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# STRENGTH CHARACTERISTICS OF COMPACTED CLAYS

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SOIL MECHANICS AND FOUNDATIONS DIVISION

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# STRENGTH CHARACTERISTICS OF COMPACTED CLAYS

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## SYNOPSIS

This paper reports the results of a laboratory investigation that was undertaken to establish the basic factors that control the shearing strength of partially saturated, compacted clays. The primary objective of the study was to isolate those factors that affected the strength and, if possible, to establish relationships that would be indicative of the relative importance of each of these variables.

Two soils were studied: one was a highly plastic clay from the Fort Union formation, which was obtained from the site of the Garrison Dam near Riverdale, North Dakota; the other was a clay of low plasticity - a modified loess from the Bluff (or Memphis) formation - that was sampled from the grounds of the Waterways Experiment Station at Vicksburg, Mississippi. Both these soils have been studied extensively by other investigators. Due to the large differences in the plasticity characteristics of the two soils, their behavior should bracket that of many clay-like soils commonly encountered.

Improved techniques were developed for preparing, compacting, and preserving soil samples that resulted in test specimens with essentially duplicate strength and stress-strain characteristics. Measurement of loads, weights, dimensions, and pressures were made with a high degree of precision. A special feature of this investigation was the determination of the void ratio of the portion of the test specimen that failed.

All specimens were approximately 1.42 inches in diameter and 2.80 inches high, and they were tested in a triaxial shear apparatus that was patterned after the one designed and built at the Soil Mechanics Laboratory, Northwestern University; certain modifications were incorporated that provided increased convenience and adaptability in testing, and greater simplicity in fabrication.

The results show conclusively that for the two soils tested, and within the ranges in water content and degree of saturation considered, the critical factor that controls the strength of partially saturated, compacted clays is the void ratio at failure. A unique relationship exists between the void ratio at failure (plotted to an arithmetic scale) and the compressive strength (plotted to a logarithmic scale). For a given set of initial conditions this relationship is independent of the confining pressure, the amount of drainage permitted, the water content, or the degree of saturation. It is strongly indicated that this relationship is valid even when the degree of saturation is increased by the addition of water from an external source.

It was found that changes in void ratio occur in partially saturated clays during the application of the shearing stresses that are not accompanied by corresponding changes in water content. The nature of these changes in void ratio is dependent upon the initial conditions, the confining pressure, the preconsolidation pressure, and the amount of water that is added to the soil

from an external source. On the basis of these facts, some of the inconsistencies in the current theories dealing with the strength characteristics of clay-like soils can be resolved.

An important by-product of this investigation was the discovery that for a given initial water content, a reduction in void ratio achieved through a process of consolidation results in a higher compressive strength than that which results if the same reduction in void ratio is achieved through a process of compaction. In addition, it was found that the amount of swell in the presence of water, at a given confining pressure, increases as the degree of compaction increases. These facts indicate that greater advantage might be taken of the beneficial effects of consolidation in the construction of earth dams and embankments than has been taken in the past. There is also a possibility that subgrades for highway and airport pavements that are moderately compacted might ultimately yield a superior supporting power than those that have been highly compacted initially. This latter statement must be tempered by other considerations, such as the effects of frost action, which still remain to be investigated.

## INTRODUCTION

In foundation and earthwork engineering - despite the heralded advances attributed to the science of soil mechanics - experience still remains the most important requisite to success. It is pertinent to note, however, that the requirements of the twentieth century are making this fact increasingly disconcerting. Earth dams of unprecedented height and size are being designed and constructed. Highway and airport pavements capable of sustaining repeated loads of ever-increasing magnitude and frequency must be built, and heavy structures are being located in areas formerly considered unsuitable from the standpoint of the supporting power of the underlying soils. In the face of these (and many other) new problems, for which suitable precedence is substantially lacking, engineers are finding it necessary to rely more and more on rational, scientific methods of analysis with correspondingly less emphasis on empirical rules.

One of the principal stumbling blocks in the application of scientific theories to engineering problems dealing with soil is the large number of factors that affect the physical behavior of soils and the great complexity of the interaction of these factors. No matter how intricate and theoretically exact the "stability analysis" of an earth structure may be, its results are likely to be misleading unless the strength characteristics of the soil, during the life of the structure, can be predicted with reasonable certainty. Extensive research (principally during the past thirty years) has developed a sound understanding of, and methods for determining within practical limits, the shearing strengths of cohesionless soils and of certain saturated, sedimentary clays. However, all compacted soils are initially partially saturated, and very little is known of the strength characteristics of these soils. As an illustration of this fact, the following is quoted from a section on the compaction of soils in one of the leading current texts on earthwork and foundation engineering[1]:

> "At the present time, the relation of moisture content at the time a fill is placed, the degree of compaction, and the physical characteristics of the fill throughout its period of service are still imperfectly

1. "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck. John Wiley and Sons, New York, 1948.

understood. The changes in strength, rigidity, and permeability of the fill with age and with changing water content deserve far more attention than they have as yet received. Hence, the remainder of this article contains almost no information concerning the properties of compacted soils. It deals primarily with construction procedures."

This paper reports the results of an attempt to contribute to our understanding of the strength characteristics of compacted clays. The development of current knowledge will be traced in some detail, though the review is in no way meant to be complete. Rather, it is intended to focus attention to the limitations of the present concepts, particularly as they apply to partially saturated, compacted clays. The results of recent research will be presented, and new concepts regarding the shearing resistance of compacted clays suggested. It is hoped that these new concepts will provide the basis for a better understanding of the behavior of compacted clays; that they will point the way to more significant field measurements, which may ultimately lead to a more rational basis for the design of compacted earth structures.

## Development of Current Knowledge

In 1776, Coulomb (1)* proposed what was probably the first formal hypothesis dealing with the frictional resistance of earth masses. This hypothesis can be expressed algebraically as follows:

$$s = \sigma \tan \phi + c$$

where,

$s$ = the maximum shearing resistance
$\sigma$ = the normal stress on any portion, a plane in an earth mass
$\phi$ = the angle of internal friction, a constant for the earth in question
$c$ = the unit cohesion, also considered to be a constant

For nearly 150 years after Coulomb's classic work was published experimentation was directed principally towards the determination of the values of $\phi$ and c for a large variety of soils. A typical example of the work done during this early period was reported by Leygue (2) in 1885, from which it was concluded that Coulomb's equation was "practically verified". Coulomb's equation soon became known as Coulomb's Law and for nearly 200 years this "Law" has dominated the thinking of engineers and soil technologists; in fact, it still remains the basis of all modern theories dealing with the stability of earth masses (3), (4).

In 1910, Frontard and Jacquinot (5) conducted experiments on samples taken from an earth dam that had failed due to a considerable lowering of the reservoir level. Insofar as the writer could determine, the conclusions from this study (as reported by Résal) were the earliest recorded statements of the fact that the unit cohesion was not a constant.

The penetrating researches and the keen observations of Dr. Karl Terzaghi planted the seeds for a better understanding of earth action (6), (7), (8), (9), (10), (11), (12). Perhaps the most significant of Terzaghi's series of researches was the formulation of a theory regarding the stress-deformation-time relationships for saturated, clay-like soils - the so-called Terzaghi theory of consolidation. Regardless of its limitations from the standpoint of predicting the settlement of structures, the theory of consolidation helped explain

* Numbers in parenthesis refer to bibliography at end of paper.

numerous phenomena experienced in foundation engineering, and it provided a basis for a more fundamental understanding of the factors responsible for the shearing resistance of soils. It drew attention to the significance of the pore-water pressure, whose magnitude is controlled by the amount of drainage (or degree of consolidation) permitted before and during a shear test. This meant that the rate of load application has an important bearing on the subsequent shearing resistance of the soil, a fact that was demonstrated by Casagrade and Albert (14) in 1930, and by the brilliant researches of Jürgenson (13) in 1934.

Between about 1912 to 1916, the Swedish railroads experienced a series of disastrous slides in which many lives were lost and property damage was extensive (15). While the main objective of this investigation was to develop reliable methods for analyzing the stability of slopes - and the method proposed by the Swedish Commission has since been widely accepted - a by-product of great significance was the discovery that when samples from a natural deposit of clay are disturbed their strength is always reduced, sometimes by as much as 90% of their undisturbed value. The recognition of this simple fact revolutionized the science and the art of soil testing (16), (17), (18). It led to a re-appraisal of the effects of sampling on test results and to the realization that certain disturbances inherent in the sampling process can never be eliminated (19). The difficulties attending the evaluation of these disturbances has led to the development of equipment for measuring shear strength in place (20). This latter approach holds promise of opening a new chapter in the art of practical soil testing, but the ever-present lack of homogeneity in natural soil deposits will probably limit its usefulness as a research tool.

From 1933 to 1936, Hvorslev (21), (22), carried out extensive investigations on the conditions for failure of saturated, remolded, cohesive soils, using both a direct shear apparatus and a torsion (or ring shear) device. Among Hvorslev's conclusions are the following:

1. A cohesive soil will [tend to] undergo, depending on its state of consolidation, either an increase or a decrease in void ratio [during shear].

2. If the rate of load application is sufficiently slow so that at any time the void ratio of the soil can be considered fully adjusted to the prevailing stress conditions, the pore-water pressure will at all times be equal to atmospheric pressure.

3. If the soil tends to reduce in volume under stresses that are applied sufficiently rapidly to prevent full adjustment of the void ratio, the pore-water pressure will be greater than atmospheric; conversely, if the soil tends to increase in volume, the pore-water pressure will be less than atmospheric.

4. The shearing resistance of saturated, remolded, cohesive soils is a function of the effective normal stress and the void ratio at the moment of failure.

5. The "true angle of internal friction" of these soils is a constant, but the cohesion is a function of the void ratio [or water content] at failure.

The first four conclusions were also arrived at independently by Rendulic (23) in 1936, using triaxial compression apparatus.

It is surprising that Hvorslev's work did not receive the recognition in this country that it so justly deserved, since it provided the basis for all subsequent research on the shearing resistance of clays. It now became clear that a given clay-like soil can exhibit a wide range in the apparent values of c and $\phi$, depending upon the amount of drainage permitted during a test. For this reason, Casagrande (24) proposed three types of tests for evaluating the shearing properties of cohesive soils: "Quick," "Consolidated-Quick," and "Slow" tests. For saturated soils these tests automatically impose certain limiting conditions of pore-water pressure, which indirectly control the magnitude of the effective stresses and of the void ratio at failure. For a given project, the particular tests that simulate the field pressure and drainage conditions must be used, and the results must be interpreted with due regard to discrepancies between the field and laboratory conditions. These facts clearly demonstrate inherent limitations to the indiscriminate application of Coulomb's Law even if the soil is initially saturated, and these limitations are particularly serious in the case of partially saturated clays.

The tremendous expansion of the work of the Corps of Engineers just prior to World War II and the unsatisfactory state of our knowledge on the physical properties of real soils at that time, prompted the Corps to authorize a cooperative research program to: (a) review pertinent seepage data, (b) conduct research on the triaxial shear method of testing soils, and, (c) conduct a soil pressure cell investigation. All three programs were started in 1940. The Waterways Experiment Station was designated as the agency to carry out the seepage and soil pressure cell studies and was instructed to act as the coordinating agency for a cooperative triaxial shear research program to be participated in by Harvard University, the Massachusetts Institute of Technology, and the Experiment Station. The results of the cooperative triaxial shear research program were presented in 17 unpublished progress reports between 1940 and 1944. In 1944, Rutledge prepared a critical review of the data from these progress reports, which was published, in 1947, by the Waterways Experiment Station (25). This report contains a wealth of information on the shearing resistance of soils and includes data on apparatus, testing techniques, and test results. The review separated cohesive soils into two groups, "homogeneous saturated clays," and "partially saturated clays." The conclusions pertinent to the present discussion are as follows:

<u>Homogeneous saturated clays</u>

1. The maximum principal stress difference ($\sigma_1 - \sigma_3$), called the compressive strength of the clay, depends only on the water content at maximum stress, provided the test specimens have not been pre-consolidated under pressures greater than the test maximum stress and then allowed to rebound to equilibrium under lower pressures. With this one limitation, it is independent of pore-water pressure and of the test method that produces the final water content.

2. A plot of water content at end of test vs. logarithm of principal stress difference is a curve that begins at the unconfined compressive strength and the natural water content and runs roughly parallel to the semi-logarithmic pressure-water content curve obtained from a standard consolidation test on the same clay.

<u>Partially saturated clays</u>

1. Four major variables affect the strength of a soil in this group.

These are: the minor principal stress, the dry soil density, the water content, and the degree of saturation.

2. For compacted soils the method of and the conditions during compaction seem to have independent effects on test strengths.

3. The relation between compressive strength and water content at end of test varies with the minor principal stress.

4. The tests in this program were not sufficient to establish the effects of all variables on the strengths of soils in this group.

In Rutledge's review, the variables effecting shearing resistance were related to the over-all strength; the use of separate components for friction and cohesion was largely abandoned. More recent work done in Europe substantiates Hvorslev's and Rutledge's conclusions but reverts back to dealing with friction and cohesion separately (26). Whatever the relative merits of the two approaches may be (of considering the individual effects of friction and cohesion or of combining them into a single strength factor) in the case of saturated clays, the results of the present investigation on compacted clays could be analyzed rationally only in terms of the over-all compressive strength.

## Soils Studied

The two clay soils selected for study differ widely in their geological origin and physical properties. One is a highly plastic clay from the Fort Union formation, which was obtained at the site of the Garrison Dam near Riverdale, North Dakota. The other is a clay of low plasticity - a modified loess from the Bluff (or Memphis) formation - that was sampled at the Waterways Experiment Station, Vicksburg, Mississippi. It is one of several used by the Experiment Station for an extensive investigation on soil compaction (27). For convenience and brevity the two soils will hereafter be referred to as "Fort Union clay" and "Mississippi loess".

### Fort Union Clay

The sedimentary rocks of the Fort Union formation were deposited during the Eocene (Tertiary) age and consist of alternate beds of sandstone, shale, coal, sandy shale, gray clay, and black carbonaceous shale (28). At the site of the Garrison Dam, the formation is approximately 475 feet thick; much of it (about 230 feet) is exposed in the bluffs along the valley of the Missouri River, but its top members are extensively covered with glacial drift. The soil samples used in this investigation come from the layers of gray clay taken from the spillway excavation at the east abutment of the dam.

In a damp condition, the Fort Union clay has a bluish-grey color that turns light grey when dried. At a moisture content corresponding to the plastic limit the soil is hard and tough. The clay has a very high dry strength but it will slake if a dried sample is immersed in water. Even at comparatively high water contents the Fort Union clay exhibits an extraordinary affinity for water. Under low confining pressures it continues to absorb water and to swell in a manner that is characteristic of the action of bentonite. From the standpoint of convenience in testing, the most unsatisfactory feature of the Fort Union clay was its extremely low permeability (about $10^{-10}$ cm./sec.,

as computed from consolidation test data). This feature not only increased the time required to perform the strength tests but also made the saturation of specimens impracticable.[2]

Mississippi Loess

The Mississippi Loess is a recent (Pleistocene) aeolian deposit about 40 miles wide that follows approximately the east bank of the Mississippi River for the entire length of the State. The loess is divisible into two parts - (a) fine, calcareous, gray to buff-colored, clayey silt, containing lime concretions, iron tubules, and a great variety of fresh-water and land shells; (b) non-calcareous yellow to brown silty clay, void of shells, but containing small iron tubules. Near Vicksburg, Natchez, and Yazoo City, cut slopes in the calcareous portion of the loess exhibit the characteristic tendency to stand vertically (30). The soil studied comes from the noncalcareous portion of the loess and was sampled from the grounds of the Waterways Experiment Station at Vicksburg, Mississippi.

In a damp condition the loess has a yellowish brown color that turns a light buff when dried. At a moisture content corresponding to the plastic limit the soil is soft and friable; accordingly, extreme care must be exercised in the preparation of test specimens to prevent spalling and other disturbances. The loess has a moderate to high dry strength and slakes very readily when a dried sample is immersed in water. At higher moisture contents the soil has very little tendency to swell when placed in contact with water; however, there seems to be a critical moisture content at which the swell becomes appreciable. This critical water content is on the dry side of Standard Proctor Optimum but its value seems to vary both with the amount and type of compaction. Under comparable conditions the loess is less compressible and more pervious than the clay. These properties reduced the time required for 100 per cent consolidation of the loess to about 1/50 that required for the clay. For this reason, despite the greater difficulties encountered in shaping test specimens, the loess was studied more extensively than the clay.

Classification data and the results of a mineralogical composition determination for the two soils are shown in Table I; grain size distribution and standard compaction curves are shown in Figures 1 and 2 respectively.

## Apparatus, Techniques and Procedures

Although the success achieved in the preparation of duplicate test specimens is considered to be an important contribution to further study of the fundamental strength characteristics of soils, space will not permit a detailed account of all the necessary techniques and procedures developed. Accordingly, only general descriptions of the apparatus and procedures used will be given here.

2. The writer found that it was not possible to saturate a compacted clay soil by any process that did not disturb the soil structure. However, the degree of saturation of a specimen could be raised more or less uniformly provided its permeability was not too low. In the case of the Fort Union clay, three weeks contact with water under pressure resulted merely in increasing the water content slightly over the lower inch of the specimen. It is for this reason that saturation of the clay was considered impracticable. At first these facts caused the writer considerable consternation but he later discovered that his experiences had been shared by others (29).

Preparation of Raw Soil

The raw soil was received at the laboratory in a disturbed condition, but most of the natural soil moisture had been retained. Upon receipt in the laboratory, it was placed in sealed metal containers, stored in a constant humidity room (relative humidity of 98%), and left in this condition for at least one month in order to obtain a reasonably constant initial water content. All further processing was carried out in the humid room.

To obtain duplicate soil specimens it is necessary to use a soil that is free of random distribution of coarse particles and that has a uniform water content and structure throughout. Many processes were tried, which included drying, mechanical disintegration, sieving, and re-wetting; various types of soil mixers, such as dough mixers and other mixers of special design; and mechanical grinders, such as meat grinders and juice extractors. None of these proved entirely satisfactory for both soils and for the ranges in water content required. Finally, a tedious and time-consuming hand method was resorted to.

Enough raw soil was removed from the metal containers to prepare all the specimens to be compacted at a given water content. If water was to be added, it was incorporated by hand kneading. The soil was then forced sieved, with the aid of a large rubber stopper, through a 1/4 inch sieve, followed by a duplicate process through a 20-mesh sieve. The fine sieve was backed with the coarse sieve for support. All visible coarse particles were removed and the product was a homogeneous material resembling finely ground coffee in appearance (Figure 3). The yield varied from 2 to 8 pounds per hour depending on the soil type and the water content.

The soil processed in this manner was placed in sealed metal containers and stored in the humid room. Each box (about 6 inches cube) contained enough soil for one compaction series, which was about 20 pounds. The maximum variation in water content between specimens of any one series was 0.2 percent (say $25.0 \pm 0.2$), and the maximum variation between specimens of different series that were intended to have the same water content was 0.4 percent.

Compaction

In the development of a compaction procedure to achieve uniformity in the initial value of the dry unit weight, advantage was taken of the experiences of Mayo (31) and dynamic compaction procedures were not considered. The compaction of individual specimens was first attempted using static pressure in a three inch diameter mold. It was found impossible to eliminate variations in unit weight at the top, center, and bottom of the specimen as a result of the side friction developed. In addition, the unavoidable variations in structure due to placing the soil into the mold for each individual specimen caused variations in the resulting stress-strain curves even when the initial conditions of void ratio and water content had been essentially duplicated. Ten inch diameter molds were then designed to minimize the effects of side friction and to permit simultaneous compaction of all the specimens to be used in a given test series.

The first mold, designed for the purpose of checking the uniformity of the dry unit weight, consisted of three rings and two collars. After compaction in this mold, the unit weight of the entire sample was determined. The variation in unit weight in a horizontal direction was evaluated by the use of a Proctor penetration needle attached to a sensitive proving frame (Figure 4). Water content determinations were made at each penetration point. The top ring was then removed, the protruding soil was cut off with a wire saw, and

the soil surface was carefully screeded with a straight edge. Again the unit weight, penetration resistances, and water contents were determined. The entire procedure was then repeated after the second ring was removed. By successive repetitions of this process a compaction procedure was developed that resulted in essentially uniform conditions of water content and void ratio.

A second ten inch diameter mold was designed to yield an "undisturbed" sample about 3½ inches high from which 21 duplicate specimens could be cut. It consisted of a split-ring section and two collars. A ten inch diameter piston and spacer block (to permit compaction from both top and bottom simultaneously) were fabricated to fit this mold with just enough clearance to eliminate friction between them and the sides of the mold yet not enough to permit the soil to enter the clearance spaces. The split-ring mold is shown assembled in Figure 5 on the platen of a 100,000 pound Southwark-Tate-Emery hydraulic testing machine in the process of compacting a test series. The maximum variation in unit weight among the 21 samples of a given series was less than 0.5 percent. This variation is roughly equal to the precision of the measurements used to determine the unit weight.

The compaction process previously described yielded a cake of soil 10 inches in diameter and 3½ inches high. The cake was transferred to the base of a circular mitre box (Figure 6), centered, and the collar (of diameter slightly larger than 10 inches) then put in place. Small pins aligned the grooves on the collar with those on the base. The soil cake was then cut along the mitred grooves by means of either a large coping saw or with piano wire. This latter process was not entirely satisfactory. A thin, continuous, high-speed band saw is considered to be the only satisfactory all-purpose tool for this purpose.

Preservation of Samples

Since the anticipated maximum time interval between compaction and shear testing exceeded six months, the use of paraffin alone as a sealing compound was not considered advisable. The work of Osterberg and Tseng (32) had shown that a mixture of 50% petrolatum and 50% paraffin was satisfactory for a period of about one year and that a total thickness of protective coating of 1/2 to 3/4 of an inch was desirable. It was feared that if such a thick coating were used the shrinkage of the paraffin might compress the samples having high void ratios. Accordingly, the following method of preservation was adopted: (a) the samples were dipped by hand until a 1/16 to 1/8 inch coat of a 50% mixture of paraffin and petrolatum was obtained; (b) each sample was then carefully wrapped in aluminum foil, care being taken to avoid air spaces between the sample and the foil; and (c) the samples were then given another 1/8 inch coat of the paraffin-petrolatum mixture, again, by hand dipping.

A control procedure was set up to determine the efficacy of the preservation process and to evaluate the effects of any thixotropic hardening that might occur. Twelve samples were selected from this series immediately after compaction and their water contents were carefully determined; six of the specimens were also tested in unconfined compression. Samples were then checked for strength and water content approximately once every month for a period of one year. The results are shown in Figure 7. These data show that the variations in strength and water content with time are random, and that they are within the precision of the process used to prepare duplicate specimens.

Preparation of Test Specimens

The test specimens used in this study were shaped, either wholly or in

part, with a power-operated soil lathe (Figure 8). All specimens were approximately 1.42 inches in diameter and 2.80 inches high. In some instances, the spindle on the lathe had to be turned by hand and the specimen shaped by successive cuts with a thin, sharp blade. The specimens prepared in this manner lacked the uniformity in shape that was achieved when the lathe was operated mechanically, but they were considerably less disturbed.

Triaxial Shear Testing

Figure 9 is a general view of the 3-bay, triaxial shear apparatus used in this study. The apparatus was patterned after the one designed and built at the Soil Mechanics Laboratory, Northwestern University (33); certain modifications were incorporated that provided increased convenience and adaptability in testing, and greater simplicity in fabrication. The unit was designed to provide an axial load of 500 kg. and a confining pressure of 10 kg. per sq. cm. at any one of the bays, each bay being completely independent with respect to confining pressure, drainage, and saturation controls. Constant strain and constant stress tests could be performed, although none of the latter were used in this study.

A single control panel (one for each bay) contains all the necessary valves and controls for evacuating, saturating, and measuring the volume change of saturated specimens. A schematic piping diagram is shown in Figure 10. The line from the air pressure cylinder permitted saturation of clay specimens by placing the water entering the bottom of the specimen under a known pressure while the top of the specimen could be subjected to a vacuum. In this manner saturation under pressure, consolidation, and shear testing could be carried out without making or breaking a single connection. After the initial difficulties with piping leaks and with alignment were overcome the precision, convenience, and adaptability of this equipment proved to be outstanding.

The effects of membrane restraint were essentially eliminated by the use of very thin rubber membranes (prophylactics). These membranes are easily ruptured and frequently contain local imperfections that may cause leakage. Due to the difficulties attending the proper calibration for membrane restraint at different confining pressures, and the importance of this effect (25), thin membranes were used for the entire study and occasional membrane failures were tolerated.

"Quick" tests were performed with the valves leading from the top and bottom of the specimen both open and closed. Tests were also run on partially consolidated specimens. When saturation was desired, the water was brought to the base of the specimen and the lower valve was closed prior to the application of the confining pressure; it was kept closed for the entire consolidation period, leaving only the top of the specimen subject to atmospheric pressure. Although the period for consolidation was increased, this procedure was necessary to prevent the air expelled during the application of the confining pressure from forming a large bubble between the water and the specimen and thereby effectively preventing the subsequent flow of water. The formation of an air bubble also prevented any attempts to saturate the specimen simultaneously with the application of the confining pressure. The water pressure used varied from 20 to 30 percent of the confining pressure; higher pressures either caused a disturbance of the soil structure or resulted in flow taking place between the specimen and the rubber membrane. As previously stated, saturation of the Fort Union clay was found to be impractical. Saturation periods of from four to fourteen days for the Mississippi

loess yielded approximately identical results, though in no case did the final degree of saturation exceed about 97 percent (see Note 2, Page 7.) A close-up of the assembled triaxial cell ready for shear testing is shown in Figure 11.

Determination of Void Ratio at Failure

In testing the soil specimens that had been loosely compacted, the observation was made that comparatively large volume changes continued to occur during the application of the deviator stress. Previous to this, all attempted correlations between strength and the initial or consolidated values of void ratio or water content - or even with the final water content - had failed. When further exhaustive tests (in which the measurement of dimensions, weights, pressures, and loads were as precise as possible using soil specimens) still failed to yield significant correlations, it was decided to attempt to measure the void ratio at maximum stress. This proved later to be a vital decision that provided the key for overcoming the difficulties and misunderstandings previously encountered.

The critical measurement in the determination of the void ratio at failure is the bulk volume of the sample. This determination was made by suspending the portion of the specimen within which failure had occurred from a very thin nylon thread (which had negligible weight and surface tension effects) and weighing it in air. The sample was then coated with a thin layer of paraffin by hand dipping, after which it was weighed in air and then when submerged in distilled water. The specific gravity of the paraffin being known, the bulk volume of the soil could be computed using Archimedes' Principle.

Once the bulk volume was obtained, the water content of the portion of the specimen that had failed was determined. The total weight, the total volume, the water content, and the specific gravity of solids being known, the void ratio at failure could be computed. The process was time consuming and beset with numerous difficulties that required considerable practice to perfect technique. In spite of this, the method was satisfactory whenever the samples failed by bulging.

When the test specimen exhibited only one well-defined failure plane, it was separated along this plane and the volume of each half was determined separately. This procedure reduced the precision of the measurement because of the smaller samples and because of irregularities along the failure plane. When the specimen exhibited more than one failure plane, the determination of the void ratio at failure by the method described above was not possible. (Fortunately, this occurred only in the unconfined compression tests, in which cases the initial void ratio was used as an approximation.) To remedy this situation a device to measure bulk volume was designed. This apparatus, though sound in principle, still remains to be perfected.

## Results

Tables 2 and 3 are summary sheets presenting the essential test results for the Fort Union clay and the Mississippi loess respectively; they contain the information from which the curves and diagrams to follow have been plotted, and from which the final conclusions were drawn.

The principal relationships obtained from the results of this study are shown in Figures 12 to 15 for the Fort Union clay, and Figures 16 to 20 for the Mississippi loess. These relationships are plots of the void ratio at failure (arithmetic scale) vs. compressive strength (logarithmic scale) - no consistent relationships were found between compressive strength and any other physical characteristic of the soil, such as water content, degree of

saturation, and the like. Corresponding to each point on these plots are shown the confining pressure, the final water content, and the final degree of saturation. For a given set of initial conditions, the relationship between compressive strength and void ratio at failure is unique - regardless of the confining pressure, the amount of drainage permitted, the water content, or the degree of saturation. This statement is certainly valid for the two soils tested within the following limits:

| | Water Content - % | Void ratio | Degree Saturation - % |
|---|---|---|---|
| Fort Union clay | 21.5 to 25.7 | 0.61 to 0.83 | 85.6 to 97.8 |
| Mississippi loess | 17.5 to 23.4 | 0.51 to 1.01 | 51.0 to 93.1 |

It is based on the results of over one hundred carefully conducted tests in which the amount of drainage permitted varied from no drainage (through a number of intermediate stages) to full drainage under confining pressures of from zero to over seven kg. per sq. cm.

Seven specimens of the Mississippi loess were placed in contact with water under a pressure of approximately 25 percent of the confining pressure for periods of time varying from 4 to 14 days. The results of all but one of these tests plotted almost exactly on the original curves. These data strongly indicate that the relationship between compressive strength and void ratio at failure is unique even when the degree of saturation is increased by the addition of water from an external source. One specimen (Figure 19) was preconsolidated at a confining pressure that exceeded its compressive strength after full rebound was permitted at a lower confining pressure. The results of this test also plotted on the above-mentioned curve.

No consistent relationships were found between stiffness (as measured by either the initial value of Young's modulus or the secant modulus to maximum stress) and any one other soil property although, in general, the strain at failure was found to increase with the confining pressure. Typical stress-strain curves for one test series are shown in Figure 21. It is believed, however, that stress-strain data are likely to be misleading so long as strains must be computed from the total deformation of the sample. A device for evaluating more closely the true strains in soils is sorely needed.

## Discussion of Results

In the early stages of the testing program two specimens were tested for each given set of test conditions with the intention of running still another test if the results from the first two specimens differed materially. It was soon found that not only did the compressive strength and the void ratio at failure for two samples tested under duplicate conditions agree very closely, in all cases, but that the corresponding stress-strain curves were also almost identical. A typical comparison is shown in Figure 22. Accordingly, only one specimen was subsequently tested under a given set of test conditions, and each point was given equal weight in plotting the curves of Figures 12 to 20 inclusive.

### Theories of Strength Variation

The most significant difference between the behavior of saturated and partially saturated clays when subjected to shear stresses is the nature of the changes in void ratio that can occur during the application of the shearing stress. It has long been known (23) that loose clays suffer a reduction in void ratio and dense clays an increase in void ratio during the application of shear stresses (in a manner entirely analogous to the behavior of sands) provided

the pore water pressure is at all times maintained equal to atmospheric pressure. When saturated clays are subjected to "Quick" or "Consolidated-Quick" tests the change in void ratio during the application of the deviator stress is negligible; any tendency towards a decrease in void ratio is counteracted by an increase in pore water pressure and, conversely, any tendency towards an increase in void ratio is counteracted by a reduction in pore water pressure. In the case of partially saturated clays, the change in void ratio during the application of the deviator stress can not only be comparatively large but it has been found to depend upon the degree of compaction and the confining pressure (Figures 23 and 24). On the basis of these simple facts the conflicting views regarding the strength characteristics of clay-like soils can be readily reconciled.

This study has shown that for a given set of initial conditions a change in compressive strength must be accompanied by a change in void ratio at failure, and that the relationship is essentially independent of the method used to bring about this change in void ratio. Thus, if a clay-like soil is initially saturated, a change in strength must be accompanied by a change in water content at failure, and the relationship between water content and compressive strength should be independent of the method used to bring about the change in water content. This latter fact has already been reported by Rutledge (25) in his review of the cooperative studies made at Harvard University and at the Massachusetts Institute of Technology. On the other hand, Rutledge concluded that the pattern of strength variation for partially saturated clays is a function of the confining pressure. This is true only insofar as the confining pressure influences the changes in void ratio that can occur during shear. A partially saturated clay can undergo a change in void ratio during shear without a corresponding change in water content, and the magnitude of this change in void ratio (for a given set of initial conditions) is a function of the confining pressure, as shown in Figures 23 and 24. Therefore, Rutledge's plot of compressive strength vs. water content at failure for partially saturated clays should be affected by the confining pressure, as his data clearly showed.

Considering now Coulomb's original approach of separating the shearing resistance of clay soils into two parts, friction and cohesion, if "Slow" shear tests are performed on a saturated clay that has never been precompressed, the Mohr rupture envelope is found to be linear and to pass through the origin. The slope of this envelope has been termed the "true angle of interval friction", which is, therefore, a constant. If a reduction in void ratio is effected without a change in intergranular pressure - for example, by preconsolidation and subsequent rebound - the shearing resistance will be increased. This increase in shearing resistance has been termed "cohesion".[3] Since a

3. The cohesion exhibited by saturated clay specimens taken from a natural soil stratum that has never been preconsolidated is considered to be due to the effects of overburden pressures that have been sustained for long periods of time. These effects are said to produce internal bonds that are equivalent to the application of external pressures, and have been called "intrinsic pressures" (3). Upon removal from the ground, a portion of the "intrinsic pressure" is lost, resulting in smaller shearing resistances as measured by laboratory tests on the best "undisturbed" samples as compared with those determined by in-place tests (19). The exact nature of the bonds produced by sustained pressures applied for long periods of time still remains to be investigated.

reduction in void ratio of a saturated soil results in a corresponding change in water content, the accompanying increase in strength, or "cohesion", should be a function of the water content. Hvorslev (21), (22), first discovered this fact in 1936, and it was substantiated recently by studies made in England (26). In the case of partially saturated clays, a change in void ratio can occur without a corresponding change in water content; hence, the representation of shearing resistance by a constant friction factor and a "cohesion" that is a function of water content is no longer valid. Relating "cohesion" to void ratio at failure, as Hvorslev originally proposed, would correct this deficiency, but the difficulties attending the evaluation of the true intergranular pressures in partially saturated clays are a serious drawback to the practical application of this approach.

It is suggested, therefore, that the pattern of strength variation of cohesive soils - saturated or unsaturated - be related to the void ratio at failure rather than the final water content, and that the maximum principal stress difference, or compressive strength, be used as a measure of the strength characteristics in lieu of the separate components of friction and cohesion.

## Compaction vs. Consolidation

The summary graphs (Figures 15 and 20) show that for a given initial water content the strength acquired at a given void ratio at failure increases as the initial degree of compaction decreases. This is indicative of the fact that the consolidation process is superior to that of compaction from the standpoint of ultimate strength. Experience has demonstrated that the central portions of embankments usually consolidate under the weight of the overburden even in those cases where they are used to impound water. Accordingly, the results of this study indicate that in the construction of earth embankments greater advantage can be taken of the beneficial effects of consolidation than has been taken in the past. While this factor may be of considerable practical importance, the reasons for the apparent superiority of the consolidation process over the compaction process is not immediately evident.

It has been demonstrated that the amount of swell exhibited by a compacted clay soil in the presence of water increases as the initial degree of compaction increases and as the confining pressure decreases (34). Since a large reduction in strength may result from a moderate amount of swell (due to the corresponding increase in void ratio), swelling is a critical factor for compacted earth structures in which the confining pressures are small. Subgrades for highway and airport pavements are typical structures of this type. It is entirely possible that high compaction coupled with subsequent swelling (after the pavement is in place) may result in an inferior subgrade to one that is only moderately compacted but in which subsequent swelling would be negligible.

The apparent advantages of consolidation over compaction in the construction of embankments and the possibilities of "overcompaction" in the case of highway and airport subgrades warrant a careful review of the current trends towards higher and higher compaction for the purpose of stabilizing these common earth structures.

## Further Research Needed

Further research is needed to substantiate the uniqueness of the compressive strength vs. void ratio at failure relationship for a variety of other soils, particularly when the degree of saturation is increased by the addition of water from an external source. The effects of wider ranges in the value of the initial water content and of consolidation under principal stress ratios

other than unity are currently being studied. The effects of preconsolidation and of other external features, such as frost action, still need to be thoroughly investigated.

From a practical standpoint there is an urgent need for a comprehensive field study to provide information on the basis of which the void ratio at failure can be predicted from the initial compacted conditions, the confining pressures, and the amount of drainage (or increase in the degree of saturation) that actually takes place over extended periods of time. This information would make it possible to predict the subsequent strength of compacted clay soils under any combination of variable external conditions - and it is only after this has been accomplished that it will be possible to formulate compaction specifications for a given job on a rational basis. In this connection a thorough investigation of the effects of different processes of compaction is mandatory, since the strength of a compacted soil at a given set of initial conditions depends upon the method as well as the amount of compaction (27).

## CONCLUSIONS

1. For the two soils tested, and within the ranges of water content, void ratio, and degree of saturation used, the pattern of strength variation for partially saturated, compacted clays can be represented by a relationship between the void ratio at failure (plotted to an arithmetic scale) and the maximum value of the principal stress difference, or compressive strength, (plotted to a logarithmic scale). For a given set of initial conditions this relationship is independent of the confining pressure, the amount of drainage permitted, the water content, or the degree of saturation. It is strongly indicated that this relationship is unique even if the degree of saturation (or water content) is increased by the addition of water from an external source, and there is a possibility that this uniqueness will hold true even after the soil has been preconsolidated and then allowed to rebound fully at a lower confining pressure.

2. The principal reason for the difference in behavior between saturated and partially saturated clays subjected to shear stresses is that in the former a change in void ratio must be accompanied by a change in water content, while in the latter, changes in void ratio can occur freely without corresponding alterations of the water content. For this reason the pattern of strength variation for saturated clays is reflected by either the water content or the void ratio at failure, while the strength variation of partially saturated clays can be related only to the void ratio at failure. On the basis of this fact the current conflicting views regarding the strength characteristics of clay-like soils can be reconciled.

3. For a given initial water content a reduction in void ratio achieved through the process of consolidation results in a higher compressive strength than that which results if the same reduction in void ratio is achieved by a process of compaction. In addition, the amount of swell in the presence of water, at a given confining pressure, increases approximately in direct proportion to the compactive effort. These facts warrant a critical review of the current trends towards higher and higher compaction for the purpose of stabilizing clay-like soils.

4. There is a need for a comprehensive field study to provide the information on the basis of which the void ratio at failure can be predicted from the initial compacted conditions, the confining pressure, the drainage conditions

and the method of compaction. Only after this has been accomplished will it be possible to formulate compaction specifications for a given job on a rational basis.

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Table I

CLASSIFICATION AND MINERALOGICAL DATA

| SOIL CHARACTERISTIC | SOIL NAME | | | |
|---|---|---|---|---|
| | Fort Union Clay | | Mississippi Loess | |
| Specific Gravity of Solids | 2.76 | | 2.72 | |
| Grain-Size Distribution | | | | |
| $D_{60}$ (mm.) | $34 \times 10^{-4}$ | | $180 \times 10^{-4}$ | |
| $D_{10}$ (mm.) | $5.4 \times 10^{-4}$ | | $15 \times 10^{-4}$ | |
| $D_{60}/D_{10}$ | 6.3 | | 12.0 | |
| Standard (Dynamic) Compaction | Standard Proctor | Modified AASHO | Standard Proctor | Modified AASHO |
| Optimum Moisture Content (%) | 25.4 | 14.5 | 18.0 | 12.8 |
| Maximum Dry Unit Weight (lb/ft$^3$) | 97.2 | 118.0 | 108.4 | 120.7 |
| Atterberg Limits | | | | |
| Liquid Limit | 70 | | 37 | |
| Plastic Limit | 26 | | 23 | |
| Plasticity Index | 44 | | 14 | |
| Mineralogical Composition | Approx. Percentage | | Approx. Percentage | |
| Quartz | 50 - 55 | | 85 - 90 | |
| Calcite | 15 | | 5 - 7 | |
| Montmorillonite | 10 - 15 | | --- | |
| Kaolinite | 10 | | --- | |
| Illite | 5 | | 4 - 5 | |
| Others | 5 | | 1 | |

## Table 2

### SUMMARY SHEET — TRIAXIAL COMPRESSION TESTS ON COHESIVE SOILS

LABORATORY OR JOB NUMBER ______ NAME OF SOIL ______ INDEX PROPERTIES, LL ___ $G_s$ ___ $D_{10}$ ___ TESTED BY G. A. Leonards

PURPOSE OF TESTS ______ DESCRIPTION OF SOIL ______ PL ___ OMC ___ $D_{60}/D_{10}$ ___ AT Purdue U.

PI ___ MD ___ DATE Dec. '50 - Aug. '51

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample Test Number and Type of Test | Initial Conditions of Specimen: Diameter $D_o$ cm | Height $H_o$ cm | Height-Diameter Ratio $H_o/D_o$ | Water Content $w_o$ % | Void Ratio $e_o$ | Dry Unit Weight $\gamma_o$ lb/ft³ | Degree of Saturation $S_o$ % | Consolidation: No. Consolidation Increments | Max. Consolidation Pressure $\sigma_3$ kg/cm² | Void Ratio after Consolidation $e_{c3}$ | Time Allowed for (Minutes): Prelim. Consol. Zero per load | Total Prelim. Consolidation | Application Axial Load: To Maximum $(\sigma_1-\sigma_3)$ | To Ultimate $(\sigma_1-\sigma_3)$ | Conditions at End of Test: Strain at Max $(\sigma_1-\sigma_3)$ $\epsilon_m$ % | Strain at End of Test $\epsilon_f$ % | Water Content $w_f$ % | Void Ratio $e_f$ | Dry Unit Weight $\gamma_f$ lb/ft³ | Degree of Saturation $S_f$ % | Compression Stresses (kg per sq cm): Minor Principal Stress $\sigma_3$ | Strength, Max $(\sigma_1-\sigma_3)$ | Maximum Major Principal Stress $\sigma_1$ | Stiffness: Initial Modulus of Deformation $E_i$ | Mod. of Deformation zero to max $(\sigma_1-\sigma_3)$ |
| **SERIES No. 1** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 2-U | 3.58 | 7.11 | 1.99 | 25.8 | .818 | 94.8 | 86.5 | — | — | — | — | — | 3.5 | 5.0 | 6.0 | 9.0 | 25.6 | — | — | — | 0 | 2.28 | 2.28 | 90 | 38.0 |
| 4-Qc | 3.60 | 7.11 | 1.97 | 25.4 | .814 | 94.9 | 86.0 | 1 | 2.11 | .759 | 1680 | 1680 | 10.0 | 10.0 | 20.0 | 22.1 | 24.5 | .736 | 99.2 | 93.6 | 2.11 | 3.72 | 5.83 | 250 | 18.6 |
| 5-Qc | 3.60 | 7.11 | 1.97 | 25.7 | .828 | 94.2 | 85.7 | 1 | 1.06 | .790 | 1230 | 1230 | 11.8 | 11.8 | 20.0 | 19.6 | 24.9 | .758 | 98.1 | 90.9 | 1.06 | 3.35 | 4.41 | 215 | 16.7 |
| 12-Qc | 3.59 | 7.11 | 1.98 | 25.3 | .815 | 94.8 | 85.7 | 1 | 3.52 | .752 | 1170 | 1170 | 10.1 | 12.0 | 16.8 | 19.9 | 24.9 | .743 | 98.8 | 92.9 | 3.52 | 3.63 | 7.15 | 345 | 21.6 |
| 13-Qc | 3.60 | 7.11 | 1.97 | 25.5 | .826 | 94.3 | 85.1 | 1 | 3.52 | .768 | 1020 | 1020 | 10.0 | 13.2 | 16.0 | 21.9 | 25.7 | .750 | 98.4 | 94.1 | 3.52 | 3.43 | 6.95 | 340 | 21.4 |
| 14-Qc | 3.61 | 7.11 | 1.97 | 25.5 | .825 | 94.4 | 85.5 | 1 | 2.11 | .777 | 960 | 960 | 11.3 | 13.0 | 18.4 | 21.1 | 25.2 | .755 | 98.2 | 91.5 | 2.11 | 3.57 | 5.68 | 250 | 18.3 |
| 15-Qc | 3.59 | 7.11 | 1.98 | 25.5 | .812 | 95.0 | 86.7 | 1 | 1.06 | .785 | 990 | 990 | 11.2 | 13.2 | 20.0 | 21.8 | 25.5 | .763 | 97.7 | 91.4 | 1.06 | 3.30 | 4.36 | 240 | 16.5 |
| 16-Qc | 3.58 | 7.11 | 1.99 | 25.5 | .824 | 94.6 | 85.3 | 1 | 0.70 | .790 | 1080 | 1080 | 14.0 | 14.9 | 19.5 | 21.0 | 25.0 | .771 | 97.3 | 89.5 | 0.70 | 3.23 | 3.93 | 235 | 16.5 |
| 17-Qc | 3.60 | 7.11 | 1.97 | 25.6 | .826 | 94.3 | 85.6 | 1 | 0.35 | .807 | 1290 | 1290 | 7.4 | 14.6 | 12.0 | 20.5 | 25.2 | .788 | 96.4 | 88.2 | 0.35 | 2.93 | 3.28 | 220 | 24.4 |
| 20-U | 3.60 | 7.11 | 1.97 | 25.6 | .826 | 94.4 | 85.5 | — | — | — | — | — | 3.6 | 4.6 | 5.8 | 7.4 | 25.3 | — | — | — | 0 | 2.15 | 2.15 | 95 | 37.1 |
| **SERIES No. 2** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1-Qc | 3.60 | 7.11 | 1.97 | 25.4 | .770 | 97.4 | 91.2 | 1 | 0.35 | .750 | 1500 | 1500 | 5.8 | 12.9 | 10.0 | 20.8 | 25.1 | .746 | 98.6 | 92.7 | 0.35 | 3.32 | 3.67 | 430 | 33.2 |
| 2-Qc | 3.60 | 7.11 | 1.97 | 25.7 | .781 | 96.8 | 90.7 | 1 | 0.70 | .759 | 1440 | 1440 | 7.2 | 13.0 | 11.8 | 21.3 | 25.1 | .740 | 98.9 | 93.4 | 0.70 | 3.36 | 4.06 | 435 | 28.5 |
| 3-U | 3.60 | 7.11 | 1.97 | 25.9 | .781 | 96.8 | 91.4 | — | — | — | — | — | 5.8 | 6.5 | 7.0 | 10.4 | 25.9 | — | — | — | 0 | 2.65 | 2.65 | 125 | 37.9 |
| 5-Qc | 3.60 | 7.11 | 1.97 | 25.6 | .772 | 97.2 | 90.8 | 1 | 2.81 | .730 | 1200 | 1200 | 8.7 | 12.4 | 14.3 | 20.1 | 25.1 | .723 | 99.9 | 95.9 | 2.81 | 3.76 | 6.57 | 580 | 26.3 |
| 6-Qc | 3.60 | 7.11 | 1.97 | 25.7 | .783 | 96.6 | 90.5 | 1 | 5.63 | .712 | 1170 | 1170 | 10.0 | 12.6 | 16.2 | 20.4 | 25.2 | .718 | 100.3 | 97.3 | 5.63 | 3.86 | 9.49 | 600 | 23.8 |
| 7-U | 3.60 | 7.11 | 1.97 | 25.6 | .776 | 96.9 | 90.6 | — | — | — | — | — | 4.2 | 6.9 | 6.8 | 11.1 | 25.9 | .780 | 96.8 | 90.3 | 0 | 2.58 | 2.58 | 125 | 38.0 |
| 8-Qc | 3.60 | 7.11 | 1.97 | 25.5 | .776 | 97.0 | 90.6 | 1 | 2.11 | .741 | 1110 | 1110 | 9.7 | 12.6 | 15.7 | 20.4 | 25.2 | .734 | 99.4 | 95.0 | 2.11 | 3.50 | 5.61 | 440 | 22.3 |
| 9-Qc | 3.63 | 7.11 | 1.96 | 25.6 | .780 | 96.8 | 90.4 | 1 | 0.35 | .760 | 1140 | 1140 | 7.1 | 11.5 | 11.5 | 18.5 | 25.5 | .756 | 98.0 | 91.9 | 0.35 | 3.22 | 3.57 | 380 | 28.0 |
| 10-Qc | 3.59 | 7.11 | 1.98 | 25.6 | .776 | 97.0 | 90.8 | 1 | 0.14 | .760 | 1080 | 1080 | 6.2 | 10.4 | 10.0 | 16.8 | 25.6 | .770 | 97.2 | 90.8 | 0.14 | 3.00 | 3.14 | 300 | 30.0 |
| 11-Qc | 3.59 | 7.11 | 1.98 | 25.6 | .777 | 97.0 | 90.1 | 1 | 2.81 | .725 | 1590 | 1590 | 10.2 | 12.7 | 16.5 | 20.6 | 24.8 | .725 | 99.9 | 94.4 | 2.81 | 3.77 | 6.58 | 440 | 22.8 |
| 12-Qc | 3.60 | 7.11 | 1.97 | 25.4 | .776 | 97.0 | 90.2 | 1 | 1.41 | .746 | 1560 | 1560 | 8.2 | 12.9 | 13.2 | 20.8 | 24.9 | .729 | 99.5 | 94.3 | 1.41 | 3.67 | 5.08 | 430 | 27.8 |
| 13-Qc | 3.59 | 7.11 | 1.98 | 25.5 | .775 | 97.0 | 90.9 | 1 | 7.04 | .680 | 2880 | 2880 | 6.8 | 10.6 | 11.1 | 17.3 | 24.6 | .693 | 101.7 | 97.2 | 7.04 | 4.41 | 11.45 | 900 | 39.8 |
| 14-Q | 3.60 | 7.11 | 1.97 | 25.6 | .776 | 97.0 | 91.0 | 1 | 1.41 | .761 | 10 | 10 | 8.2 | 8.2 | 10.0 | 10.0 | 25.6 | .752 | 98.3 | 94.0 | 1.41 | 3.29 | 4.70 | 415 | 32.9 |
| 15-Qc | 3.60 | 7.11 | 1.97 | 25.6 | .772 | 97.2 | 91.2 | 1 | 5.63 | .694 | 2700 | 2700 | 8.1 | 11.6 | 13.1 | 18.9 | 24.6 | .704 | 101.0 | 96.4 | 5.63 | 4.12 | 9.75 | 660 | 31.5 |
| 13A-Qc | 3.60 | 7.11 | 1.97 | 25.4 | .776 | 97.0 | 90.4 | 1 | 2.81 | .693 | 7140 | 7140 | 10.0 | 11.8 | 16.0 | 19.6 | 23.5 | .674 | 102.8 | 96.2 | 2.81 | 5.00 | 7.81 | 905 | 31.2 |
| **SERIES No. 4** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1-U | 3.60 | 7.11 | 1.97 | 25.6 | .746 | 98.7 | 94.7 | — | — | — | — | — | 10.8 | 12.4 | 19.2 | 20.0 | 25.6 | — | — | — | 0 | 2.79 | 2.79 | 110 | 18.3 |
| 2-Qc | 3.60 | 7.11 | 1.97 | 25.6 | .747 | 98.6 | 94.6 | 1 | 0.21 | .727 | 1470 | 1470 | 9.5 | 10.8 | 9.5 | 17.4 | 25.0 | .722 | 100.0 | 95.4 | 0.21 | 3.46 | 3.67 | 350 | 36.4 |
| 3-Qc | 3.60 | 7.11 | 1.97 | 25.5 | .747 | 98.6 | 94.3 | 1 | 0.70 | .718 | 1500 | 1500 | 7.6 | 11.6 | 11.8 | 18.7 | 24.8 | .711 | 100.6 | 96.1 | 0.70 | 3.64 | 4.34 | 570 | 30.9 |
| 4-Qc | 3.60 | 7.11 | 1.97 | 25.5 | .743 | 98.8 | 94.5 | 1 | 2.81 | .709 | 1470 | 1470 | 7.5 | 12.2 | 11.9 | 19.8 | 24.8 | .708 | 100.9 | 96.8 | 2.81 | 3.67 | 6.48 | 570 | 30.8 |
| 5-U | 3.60 | 7.11 | 1.97 | 25.5 | .743 | 98.8 | 94.6 | — | — | — | — | — | 9.8 | 11.7 | 15.5 | 18.8 | 25.5 | .744 | 98.8 | 94.8 | 0 | 2.80 | 2.80 | 115 | 18.4 |
| 6-Q | 3.60 | 7.11 | 1.97 | 25.4 | .740 | 99.0 | 94.7 | 1 | 7.04 | .696 | 10 | 10 | 7.1 | 11.7 | 11.5 | 19.0 | 25.3 | .714 | 100.4 | 97.7 | 7.04 | 3.55 | 10.59 | 570 | 30.9 |
| 7-Qc | 3.60 | 7.11 | 1.97 | 25.4 | .740 | 99.0 | 94.7 | 1 | 1.41 | .701 | 2880 | 2880 | 5.8 | 12.0 | 9.5 | 19.5 | 24.3 | .692 | 101.8 | 96.9 | 1.41 | 4.05 | 5.46 | 630 | 42.5 |
| 8-Qc | 3.60 | 7.11 | 1.97 | 25.6 | .746 | 98.7 | 94.5 | 1 | 5.63 | .683 | 1380 | 1380 | 6.1 | 13.0 | 9.9 | 21.2 | 24.9 | .697 | 101.5 | 98.6 | 5.63 | 3.83 | 9.46 | 770 | 38.7 |
| 9-Q | 3.60 | 7.11 | 1.97 | 25.4 | .743 | 98.8 | 94.4 | 1 | 1.41 | .727 | 10 | 10 | 7.6 | 7.6 | 12.2 | 12.3 | 25.3 | .727 | 99.7 | 96.1 | 1.41 | 3.31 | 4.72 | 330 | 27.2 |
| 10-Q | 3.60 | 7.11 | 1.97 | 25.5 | .744 | 98.8 | 94.5 | 1 | 2.81 | .727 | 10 | 10 | 7.7 | 8.0 | 12.0 | 12.9 | 25.4 | .725 | 99.8 | 96.4 | 2.81 | 3.32 | 6.13 | 325 | 27.7 |
| 11-Qc | 3.60 | 7.11 | 1.97 | 25.7 | .744 | 98.8 | 95.3 | 1 | 4.22 | .678 | 5760 | 5760 | 7.0 | 10.9 | 11.4 | 17.7 | 24.6 | .689 | 102.0 | 98.5 | 4.22 | 4.16 | 8.38 | 850 | 36.5 |
| 12-Qc | 3.60 | 7.11 | 1.97 | 25.4 | .744 | 98.8 | 94.3 | 1 | 2.81 | .666 | 6780 | 6780 | 9.9 | 9.6 | 15.5 | 15.6 | 23.4 | .665 | 103.5 | 97.3 | 2.81 | 5.00 | 7.81 | 900 | 30.7 |
| 11A-Qc | 3.60 | 7.11 | 1.97 | 25.2 | .744 | 98.8 | 93.7 | 1 | 5.63 | .677 | 3180 | 3180 | 6.4 | 10.7 | 11.0 | 17.2 | 24.7 | .688 | 102.0 | 98.7 | 5.63 | 4.23 | 9.86 | 850 | 38.4 |
| 11B-Qc | 3.60 | 7.11 | 1.97 | 25.5 | .747 | 98.7 | 94.5 | 1 | 2.81 | .618 | | 15,000 | 7.9 | 11.7 | 11.8 | 18.7 | 21.5 | .607 | 107.2 | 97.8 | 2.81 | 7.82 | 10.63 | 1400 | 66.2 |

27: Sketch of Failure

28: Type of Testing Equipment: constant strain - piston type - glycerine as pressure fluid

Membrane strength correction of 0 kg per sq cm deducted from these results

REMARKS

Very thin membranes (0.0025" thick) used in these tests.

* No final void ratio determinations due to excessive cracking of specimens.

## Table 3. Mississippi Loess.

### SUMMARY SHEET — TRIAXIAL COMPRESSION TESTS ON COHESIVE SOILS

LABORATORY OR JOB NUMBER ______ NAME OF SOIL ______ INDEX PROPERTIES, LL ___ $G_s$ ___ $D_{10}$ ___ TESTED, BY G. A. Leonards
PURPOSE OF TESTS ______ DESCRIPTION OF SOIL ______ PL ___ OMC ___ $D_{60}/D_{10}$ ___ AT Purdue U.
PI ___ MD ___ DATE Dec. '50 - Aug. '51

| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Sample Test Number and Type of Test | Initial Conditions of Specimen: Diameter | Height | Height-Diameter Ratio | Water Content | Void Ratio | Dry Unit Weight | Degree of Saturation | Consolidation: No. Consolidation Increments | Max. Consolidation Pressure | Void Ratio after Consolidation | Time Allowed for: Prelim. Consol. Inc. per Incr. | Total Prelim. Consolidation | Application Axial Load: To Maximum $(\sigma_1-\sigma_3)$ | To Ultimate $(\sigma_1-\sigma_3)$ | Conditions at End of Test: Strain at Max. $(\sigma_1-\sigma_3)$ | Strain at End of Test | Water Content | Void Ratio | Dry Unit Weight | Degree of Saturation | Compression Stresses: Minor Principal Stress | STRENGTH, MAX $(\sigma_1-\sigma_3)$ | Maximum Major Principal Stress | Stiffness: Initial Modulus of Deformation | Mod. of Deformation zero to max $(\sigma_1-\sigma_3)$ | Sketch of Failure |
| | $D_o$ cm | $H_o$ cm | $H_o/D_o$ | $w_o$ % | $e_o$ | $\gamma_o$ lb/ft³ | $S_o$ % | | $\sigma_3$ kg/cm² | $e_c$ | Minutes | | | | $\epsilon_m$ % | $\epsilon_e$ % | $w_e$ % | $e_e$ | $\gamma_e$ lb/ft³ | $S_e$ % | $\sigma_3$ kg per sq cm | $\Delta\sigma$ | $\sigma_1$ | $E_o$ | $E_m$ | |
| **SERIES No. 1** | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 2-U | 3.59 | 7.11 | 1.98 | 18.7 | 1.003 | 84.6 | 50.9 | — | — | — | — | — | 1.6 | 2.8 | 2.6 | 4.5 | 18.7 | — | — | — | 0 | 0.49 | 0.49 | 11 | 16.9 | |
| 3-U | 3.60 | 7.11 | 1.97 | 18.6 | 0.989 | 85.3 | 51.2 | — | — | — | — | — | 1.4 | 2.2 | 2.1 | 3.9 | 18.6 | — | — | — | 0 | 0.51 | 0.51 | 33 | 22.2 | |
| 4-Q | 3.59 | 7.11 | 1.98 | 18.5 | 0.994 | 85.0 | 50.5 | 1 | 0.35 | 0.969 | 10 | 10 | 11.7 | 11.7 | 20.0 | 18.9 | 18.5 | 0.870 | 90.5 | 57.8 | 0.35 | 1.43 | 1.78 | 150 | 7.1 | |
| 6-Qc | 3.59 | 7.11 | 1.98 | 18.4 | 0.987 | 85.4 | 50.6 | 1 | 1.41 | 0.910 | 3180 | 3180 | 12.0 | 12.0 | 20.0 | 19.5 | 18.4 | 0.674 | 101.9 | 74.8 | 1.41 | 3.60 | 5.01 | 325 | 18.0 | |
| 7-Qc | 3.59 | 7.11 | 1.98 | 18.4 | 0.988 | 85.4 | 50.7 | 1 | 2.81 | 0.842 | 2880 | 2880 | 11.6 | 11.6 | 20.0 | 19.1 | 18.1 | 0.589 | 107.2 | 84.8 | 2.81 | 5.46 | 8.27 | 600 | 27.3 | |
| 8-Qc | 3.59 | 7.11 | 1.98 | 18.4 | 0.983 | 85.6 | 51.1 | 1 | 5.63 | 0.716 | 2700 | 2700 | 11.9 | 11.9 | 20.0 | 20.1 | 18.0 | 0.569 | 109.5 | 86.8 | 5.63 | 6.90 | 12.53 | 1600 | 34.9 | |
| 9-Qc | 3.58 | 7.11 | 1.99 | 18.4 | 1.003 | 84.6 | 49.9 | 1 | 0.35 | 1.000 | 1740 | 1740 | 12.3 | 12.3 | 20.0 | 19.8 | 18.1 | 0.867 | 90.8 | 56.6 | 0.35 | 1.45 | 1.80 | 180 | 7.2 | |
| 10-Qc | 3.59 | 7.11 | 1.98 | 18.4 | 1.010 | 84.4 | 49.5 | 1 | 0.70 | 1.009 | 1350 | 1350 | 12.3 | 12.3 | 20.0 | 19.3 | 18.7 | 0.753 | 96.8 | 67.6 | 0.70 | 2.26 | 2.96 | 220 | 11.3 | |
| 11-Q | 3.60 | 7.11 | 1.97 | 18.4 | 0.990 | 85.3 | 50.7 | 1 | 0.21 | 0.981 | 10 | 10 | 12.2 | 12.2 | 20.0 | 19.7 | 18.6 | 0.899 | 89.4 | 56.2 | 0.21 | 1.18 | 1.39 | 80 | 5.9 | |
| **SERIES No. 2** | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1-U | 3.60 | 7.11 | 1.97 | 18.6 | 0.766 | 97.5 | 68.3 | — | — | — | — | — | 3.1 | 4.2 | 5.0 | 6.8 | — | — | — | — | 0 | 1.32 | 1.32 | 60 | 26.4 | • |
| 2-U | 3.59 | 7.11 | [illegible] | 18.5 | 0.742 | 97.3 | 67.6 | — | — | — | — | — | 3.2 | 4.3 | 4.8 | 6.4 | — | — | — | — | 0 | 1.29 | 1.29 | 70 | 26.8 | • |
| 3-Qc | 3.59 | 7.11 | 1.98 | 18.1 | 0.730 | 98.1 | 67.2 | 1 | 0.70 | 0.702 | 1320 | 1320 | 11.1 | 12.6 | 18.0 | 20.4 | 17.9 | 0.674 | 101.2 | 70.0 | 0.70 | 3.05 | 3.75 | 380 | 17.0 | |
| 4-Qc | 3.59 | 7.11 | 1.98 | 18.1 | 0.739 | 97.6 | 66.6 | 1 | 1.41 | 0.707 | 1200 | 1200 | 11.9 | 12.5 | 20.0 | 20.1 | 17.9 | 0.619 | 104.8 | 77.9 | 1.41 | 4.20 | 5.61 | 480 | 21.0 | |
| 5-Qc | 3.59 | 7.11 | 1.98 | 18.0 | 0.739 | 97.8 | 66.6 | 1 | 2.81 | 0.699 | 720 | 720 | 11.8 | 11.8 | 20.0 | 19.1 | 17.8 | 0.581 | 107.3 | [illegible] | 2.81 | 5.24 | 8.05 | 1100 | 26.2 | |
| 6-Q | 3.59 | 7.11 | 1.98 | 18.1 | 0.730 | 98.1 | 67.6 | 1 | 0.35 | 0.719 | 10 | 10 | 7.9 | 10.8 | 12.0 | 17.4 | 18.1 | 0.713 | 99.0 | [illegible] | 0.35 | 2.26 | 2.61 | 170 | 18.8 | |
| 7-Qc | 3.59 | 7.11 | 1.98 | 18.1 | 0.733 | 98.0 | 67.2 | 1 | 5.63 | 0.654 | 190 | 190 | 11.6 | 11.6 | 20.0 | 18.9 | 17.9 | 0.565 | 108.8 | 86.4 | 5.63 | 6.02 | 11.65 | 1450 | 30.1 | |
| 9-Qc | 3.59 | 7.11 | 1.98 | 18.1 | 0.734 | 97.9 | 66.9 | 1 | 0.85 | 0.715 | 240 | 240 | 12.2 | 12.2 | 20.0 | 19.7 | 17.9 | 0.667 | 103.3 | 75.4 | 0.85 | 3.37 | 4.22 | 270 | 16.9 | |
| **SERIES No. 3** | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1-U | 3.61 | 7.11 | 1.97 | 18.5 | .580 | 107.2 | 86.6 | — | — | — | — | — | 5.8 | 6.7 | 9.5 | 11.8 | 18.4 | — | — | — | 0 | 2.77 | 2.77 | 80 | 29.2 | • |
| 2-U | 3.61 | 7.11 | 1.97 | 18.5 | .579 | 107.2 | 86.3 | — | — | — | — | — | 5.2 | 6.0 | 8.5 | 9.7 | 18.4 | — | — | — | 0 | 3.00 | 3.00 | 90 | 35.2 | • |
| 3-Qc | 3.61 | 7.11 | 1.97 | 18.0 | .568 | 108.2 | 86.3 | 1 | 2.81 | .547 | 560 | 560 | 11.3 | 11.3 | 20.0 | 18.4 | 17.9 | .552 | 109.4 | 88.1 | 2.81 | 7.05 | 9.86 | 650 | 35.3 | |
| 4-Qc | 3.61 | 7.11 | 1.97 | 18.2 | .567 | [illegible] | 87.0 | 1 | 1.41 | .566 | 360 | 360 | 9.9 | 12.3 | 16.0 | 19.9 | 17.9 | .567 | 108.4 | 86.5 | 1.41 | 5.54 | 6.95 | 350 | 34.5 | |
| 5-Qc | 3.61 | 7.11 | 1.97 | 18.0 | .568 | 108.3 | 86.4 | 1 | 0.70 | .569 | 240 | 240 | 8.3 | 8.3 | 14.0 | 16.0 | 18.1 | .578 | 107.4 | 84.0 | 0.70 | 4.40 | 5.10 | 220 | 31.4 | |
| 6-Qc | 3.60 | 7.11 | 1.98 | 18.3 | .576 | 107.7 | 86.2 | 1 | 1.41 | .557 | 480 | 480 | 10.5 | 12.4 | 17.0 | 20.1 | 18.0 | .571 | 108.0 | 85.5 | 1.41 | 5.39 | 6.80 | 350 | 31.7 | |
| 7-Qc | 3.60 | 7.11 | 1.98 | 18.3 | .575 | 107.9 | 86.6 | 1 | 5.63 | .520 | 120 | 120 | 12.0 | 12.0 | 20.0 | 19.9 | 17.9 | .539 | 110.3 | 90.4 | 5.63 | 8.94 | 14.57 | 1350 | 44.1 | |
| 9-Qc | 3.61 | 7.11 | 1.97 | 18.2 | .571 | 108.0 | 86.7 | 1 | 7.04 | .509 | 180 | 180 | 11.8 | 11.8 | 20.0 | 19.3 | 17.7 | .531 | 110.8 | 90.5 | 7.04 | 10.49 | 17.53 | 1400 | 52.5 | |
| 10-Qc | 3.59 | 7.11 | 1.98 | 18.1 | .564 | 108.5 | 87.2 | 1 | 1.41 | .560 | 300 | 300 | 11.7 | 11.7 | 20.0 | 19.0 | 19.1 | .577 | 107.5 | 90.1 | 1.41 | 4.68 | 6.09 | 90 | 23.4 | * |
| 11-Qc | 3.60 | 7.11 | 1.98 | 17.9 | .562 | 108.6 | 86.6 | 1 | 0.70 | .554 | 120 | 120 | 11.4 | 11.4 | 20.0 | 18.9 | 19.4 | .587 | 106.8 | 90.0 | 0.70 | 3.62 | 4.32 | 120 | 18.1 | * |
| **SERIES No. 5** | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1-U | 3.61 | 7.11 | 1.97 | 22.7 | .738 | 97.7 | 84.0 | — | — | — | — | — | 6.0 | 9.5 | 9.6 | 15.3 | 22.9 | — | — | — | 0 | 0.86 | 0.86 | 70 | 9.0 | * |
| 2-U | 3.59 | 7.11 | 1.98 | 22.8 | .739 | 97.6 | 83.9 | — | — | — | — | — | 6.4 | 9.0 | 10.4 | 14.9 | 22.9 | — | — | — | 0 | 0.85 | 0.85 | 90 | 8.2 | * |
| 3-Qc | 3.59 | 7.11 | 1.98 | 22.8 | .741 | 97.5 | 83.5 | 1 | 0.70 | .716 | 1020 | 1020 | 12.6 | 12.5 | 20.0 | 20.1 | 23.4 | .709 | 99.3 | 89.7 | 0.70 | 1.32 | 2.02 | 230 | 6.6 | |
| 4-Qc | 3.59 | 7.11 | 1.98 | 22.8 | .737 | 97.7 | 84.0 | 1 | 1.41 | .704 | 1080 | 1080 | 11.7 | 11.7 | 20.0 | 18.9 | 22.4 | .671 | 101.6 | 90.9 | 1.41 | 1.94 | 3.35 | 540 | 9.7 | |
| 5-Qc | 3.60 | 7.11 | 1.98 | 22.9 | .744 | 97.3 | 83.5 | 1 | 2.81 | .682 | 900 | 900 | 11.9 | 11.9 | 20.0 | 19.4 | 21.5 | .640 | 103.5 | 91.4 | 2.81 | 2.89 | 5.70 | 850 | 14.5 | |
| 6-Qc | 3.59 | 7.11 | 1.98 | 22.8 | .737 | 97.7 | 84.1 | 1 | 1.06 | .709 | 760 | 760 | 12.1 | 12.1 | 20.0 | 19.6 | 22.8 | .688 | 100.6 | 90.1 | 1.06 | 1.62 | 2.68 | 330 | 8.1 | |
| 7-Qc | 3.58 | 7.11 | 1.99 | 22.8 | .747 | 97.2 | 83.0 | 1 | 2.11 | .699 | 660 | 660 | 11.8 | 11.8 | 20.0 | 19.1 | 21.9 | .657 | 102.4 | 90.7 | 2.11 | 2.32 | 4.43 | 550 | 11.6 | |
| 8-Qc | 3.59 | 7.11 | 1.98 | 22.8 | .745 | 97.3 | 83.2 | 1 | 5.63 | .653 | 480 | 480 | 11.0 | 11.0 | 20.0 | 18.6 | 20.7 | .603 | 105.9 | 93.1 | 5.63 | 4.71 | 10.34 | 1100 | 23.6 | |
| 9-Qc | 3.59 | 7.11 | 1.98 | 22.9 | .748 | 97.1 | 83.1 | 1 | 7.04 | .634 | 420 | 420 | 11.5 | 11.5 | 20.0 | 19.5 | 19.9 | .585 | 107.1 | 92.6 | 7.04 | 5.55 | 12.59 | 1350 | 27.8 | |
| 10-Qc | 3.60 | 7.11 | 1.98 | 22.7 | .746 | 97.2 | 82.8 | 1 | 4.22 | .667 | 1020 | 1020 | 11.4 | 11.3 | 20.0 | 19.0 | 21.0 | .616 | 105.0 | 92.4 | 4.22 | 3.72 | 7.94 | 950 | 18.6 | |
| 11-Q | 3.60 | 7.11 | 1.98 | 22.9 | .746 | 97.2 | 83.5 | — | — | — | — | — | 12.1 | 12.1 | 20.0 | 19.6 | 22.5 | .676 | 101.3 | 90.7 | 2.81 | 1.82 | 4.63 | 500 | 9.1 | |
| 12-Qc | 3.59 | 7.11 | 1.98 | 22.8 | .745 | 97.2 | 83.2 | 1 | 1.06 | .714 | 600 | 600 | 11.5 | 11.5 | 20.0 | 18.7 | 23.5 | .710 | 99.2 | 90.0 | 1.06 | 1.34 | 2.40 | 400 | 6.7 | * |
| 13-Qc | 3.59 | 7.11 | 1.98 | 22.8 | .739 | 97.6 | 83.7 | 1 | 2.11 | .700 | 480 | 480 | 11.8 | 11.8 | 20.0 | 19.4 | 22.7 | .676 | 101.3 | 91.3 | 2.11 | 1.91 | 4.02 | 850 | 9.6 | * |
| 14-Qc | 3.57 | 7.11 | 1.99 | 22.9 | .744 | 97.3 | 83.7 | 1 | 4.22 | .667 | 440 | 440 | 11.8 | 11.8 | 20.0 | 19.8 | 20.9 | .620 | 104.8 | 91.7 | 4.22 | 3.62 | 7.84 | 1100 | 18.1 | * |
| 15-Qc | 3.58 | 7.11 | 2.00 | 22.7 | .748 | 97.6 | 83.4 | 2 | 4.22 / 1.41 | .661 / .700 | 630 | 1260 | 11.4 | 11.4 | 20.0 | 18.9 | 21.0 | .634 | 103.9 | 90.2 | 1.41 | 2.98 | 4.39 | 700 | 14.9 | * |

28 — Type of Testing Equipment: constant strain - piston type - glycerine as pressure fluid

Membrane strength correction of 0 kg per sq cm deducted from these results

REMARKS

* NO FINAL VOID RATIO DETERMINATIONS DUE TO EXCESSIVE CRACKING OF SPECIMENS.

• AFTER CONSOLIDATION, FULL CONFINING PRESSURE WAS MAINTAINED AND THE SAMPLE SUBJECTED TO WATER UNDER PRESSURE FROM 4 TO 14 DAYS

* PRECONSOLIDATED, THEN ALLOWED TO REBOUND FULLY UNDER LOWER CONFINING PRESSURE

Figure 1

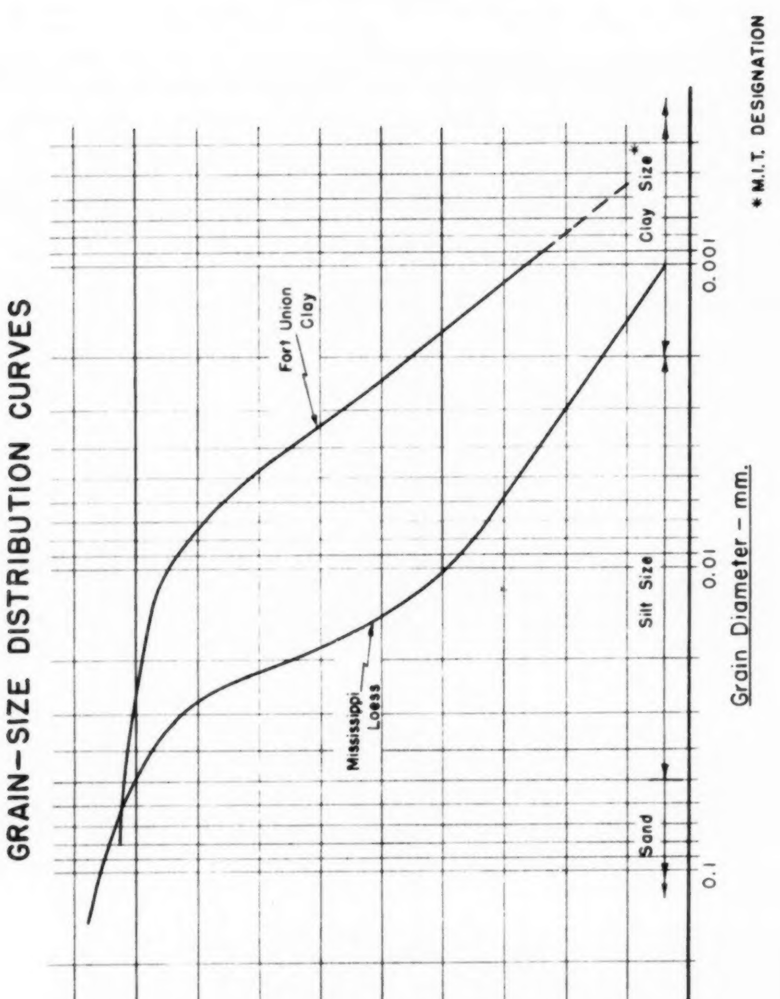
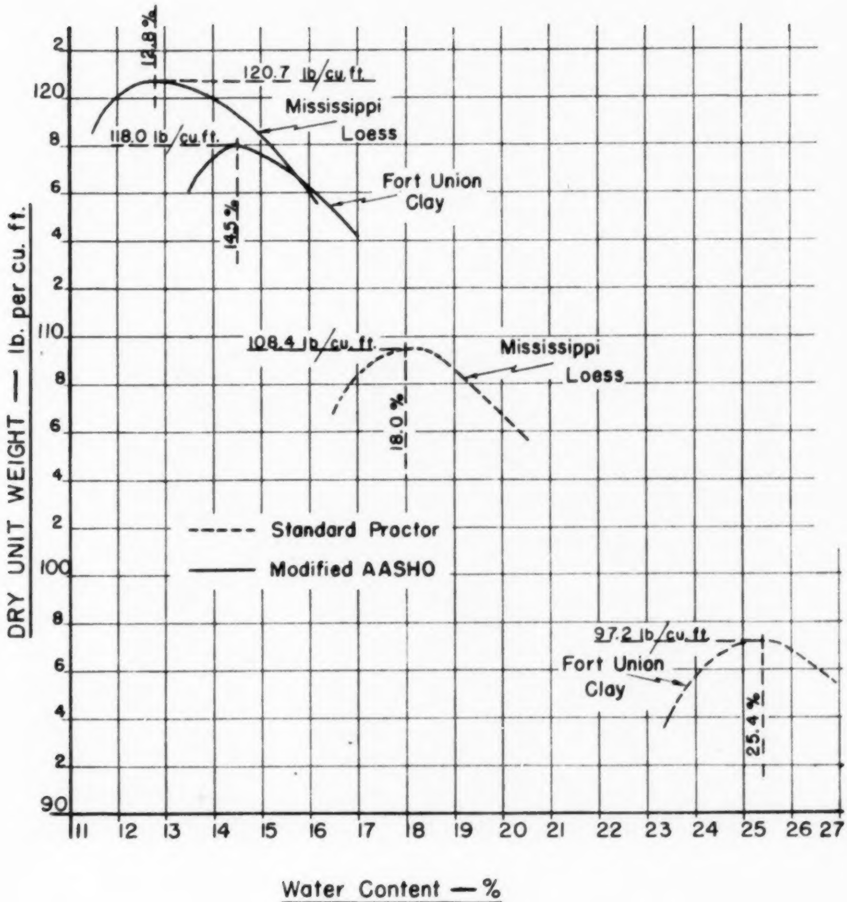

GRAIN-SIZE DISTRIBUTION CURVES
Fort Union Clay
Mississippi Loess
% By Weight Finer Than
100
90
80
70
60
50
40
30
20
10
0
Sand
Silt Size
Clay Size*
0.1
0.01
0.001
Grain Diameter – mm.
* M.I.T. DESIGNATION

Figure 2

## STANDARD COMPACTION CURVES



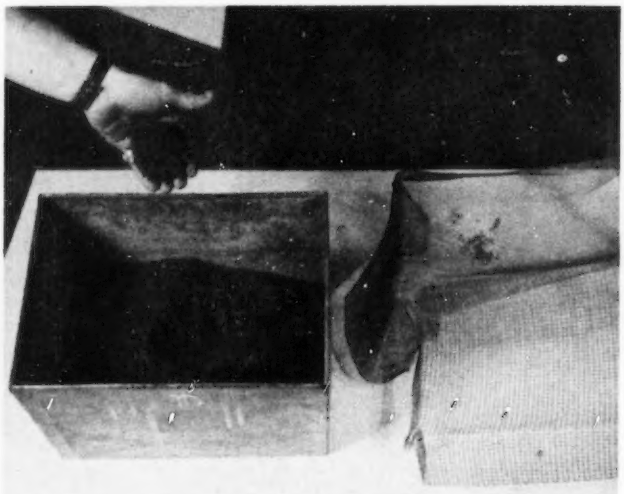


Fig. 3. Processing Raw Soil by Forced Sieving - and the Product.

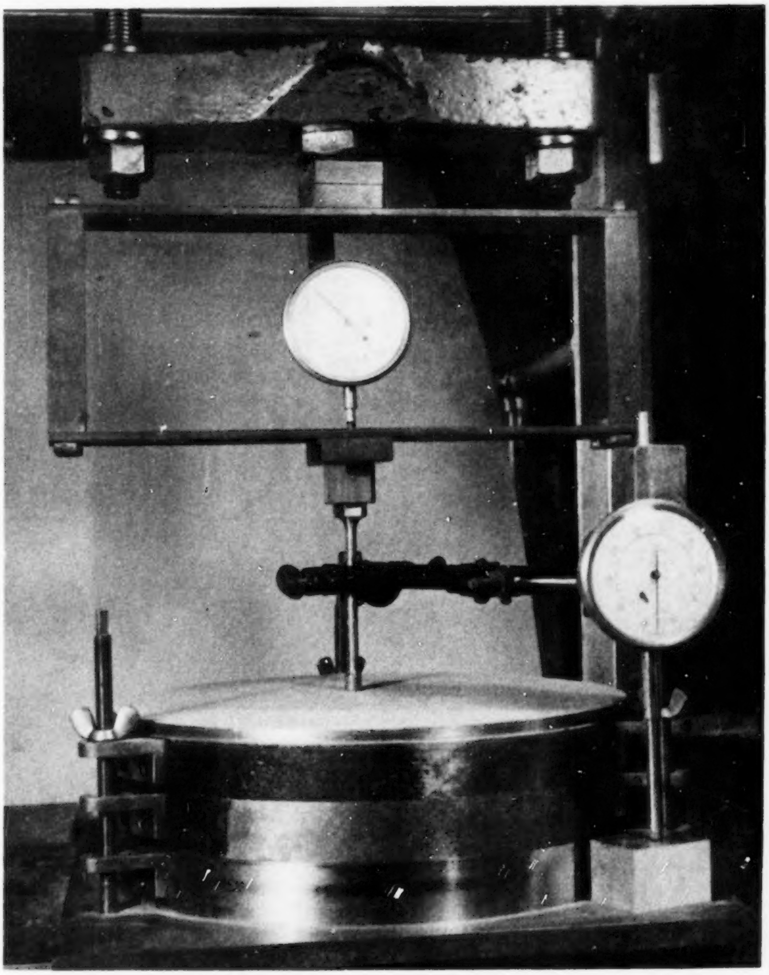


Fig. 4. Proctor Needle and Proving Frame Used to Evaluate the Variation in Unit Weight in a Horizontal Direction.

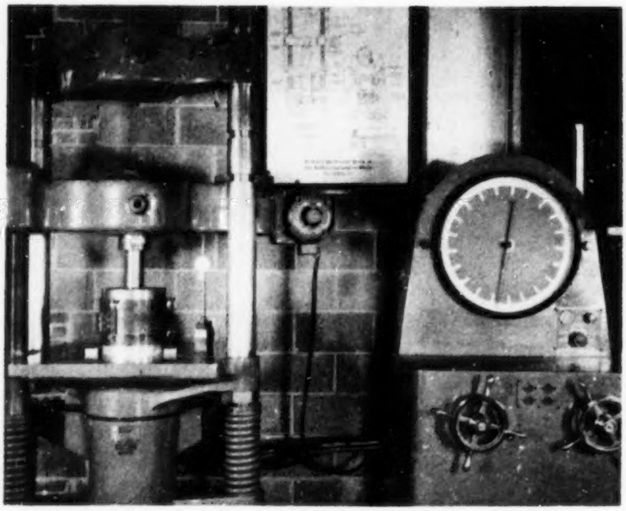


Fig. 5. General View of Compaction Equipment.

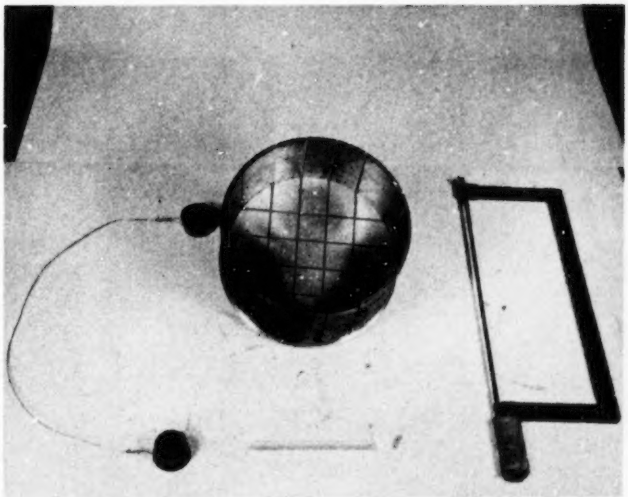
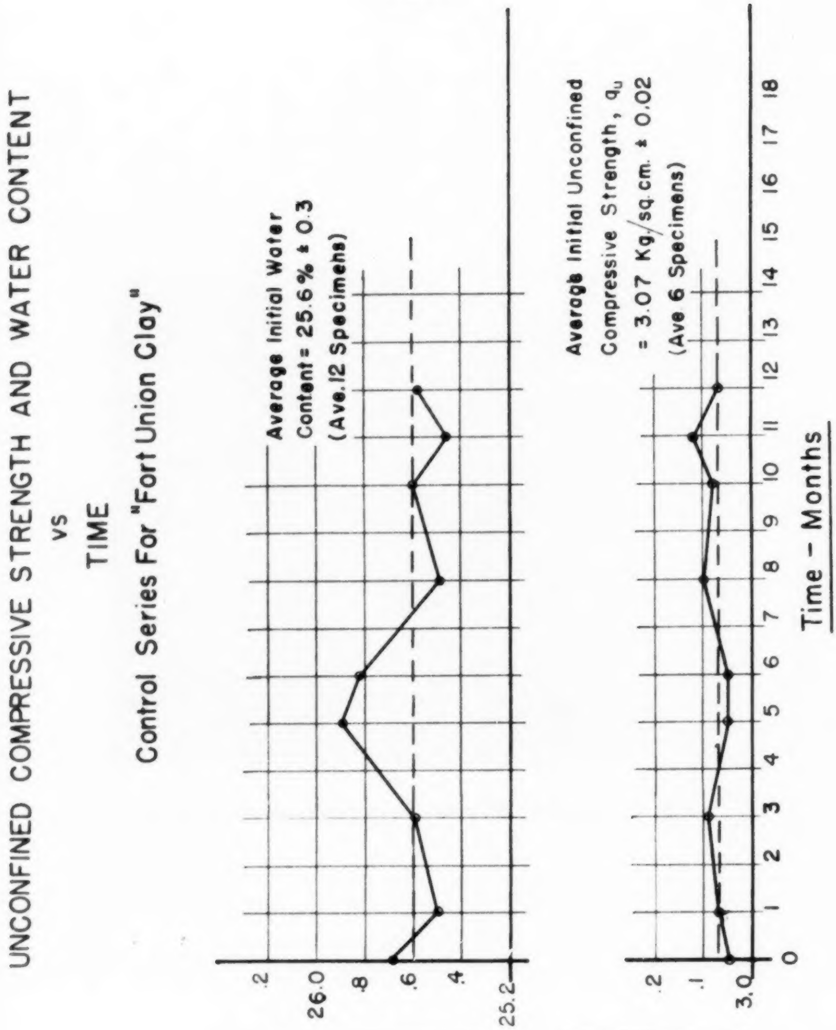


Fig. 6. Circular Mitre Box and Typical Cutting Tools.

Figure 7



UNCONFINED COMPRESSIVE STRENGTH AND WATER CONTENT
vs
TIME
Control Series For "Fort Union Clay"
Average Initial Water Content = 25.6% ± 0.3 (Ave. 12 Specimens)
Average Initial Unconfined Compressive Strength, $q_u$ = 3.07 Kg./sq.cm. ± 0.02 (Ave 6 Specimens)
Water Content - %
.2
26.0
.8
.6
.4
25.2
$q_u$ - kg./sq.cm.
.2
.1
3.0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Time - Months

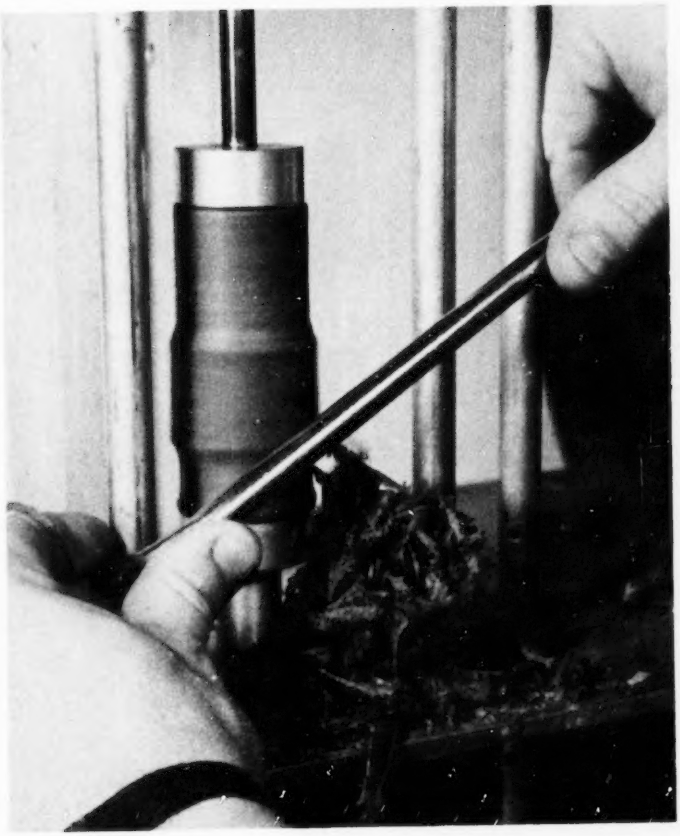


Fig. 8. Trimming a Specimen in the Soil Lathe.

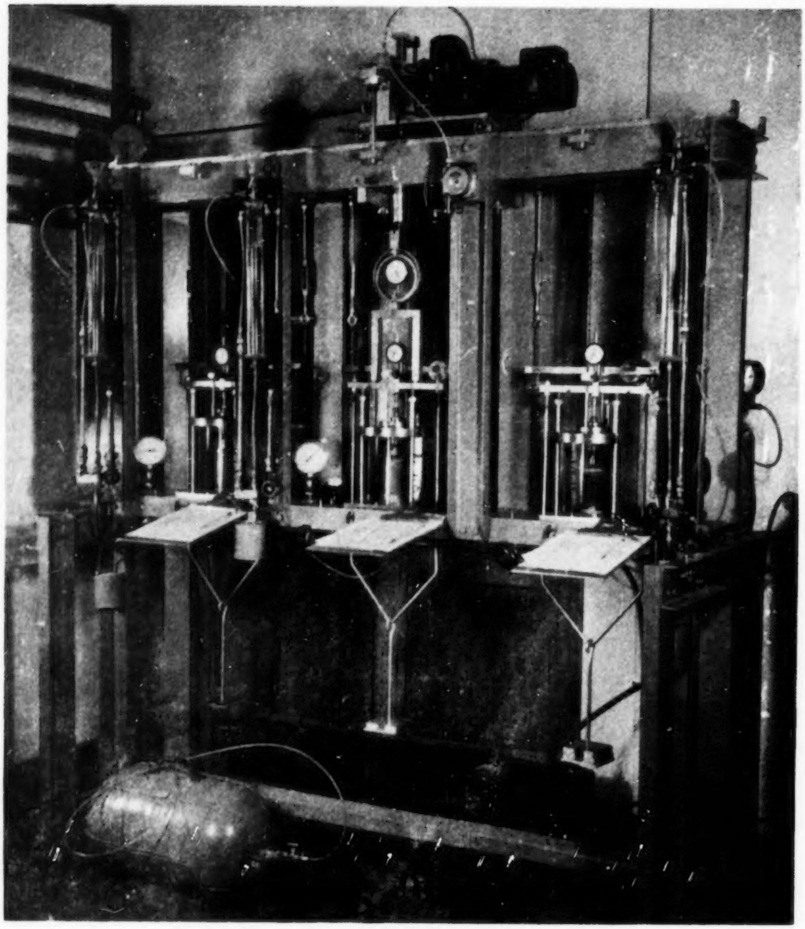
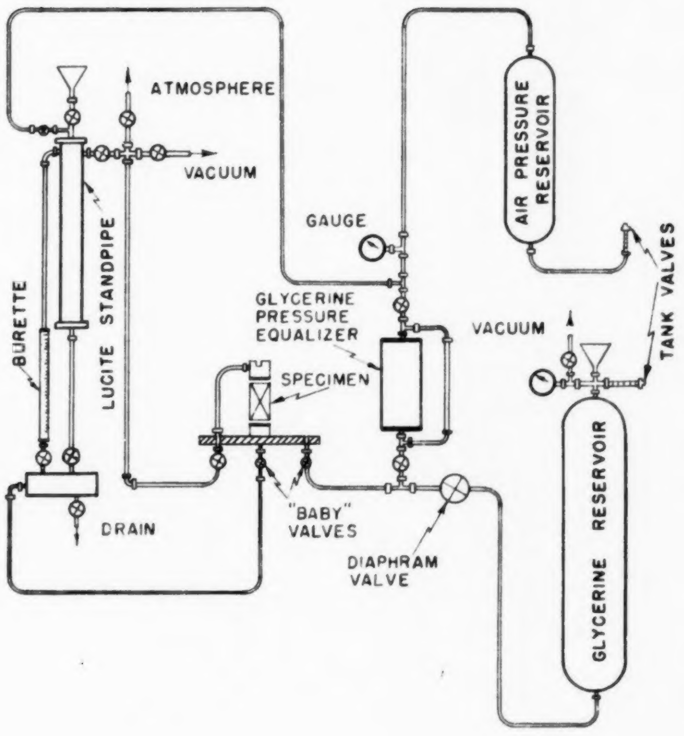


Fig. 9. General View of Three-Bay Triaxial Testing Apparatus.

Figure 10

# SCHEMATIC PIPING DIAGRAM
# TRIAXIAL COMPRESSION APPARATUS




Not To Scale

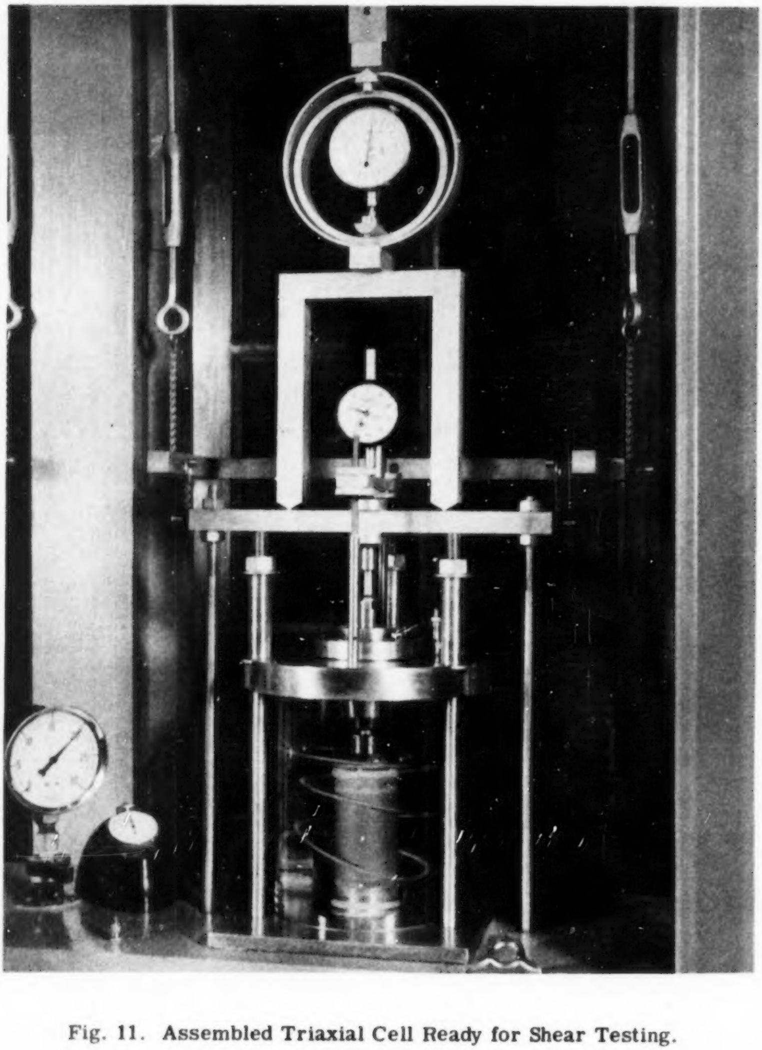


Fig. 11. Assembled Triaxial Cell Ready for Shear Testing.

Figure 12

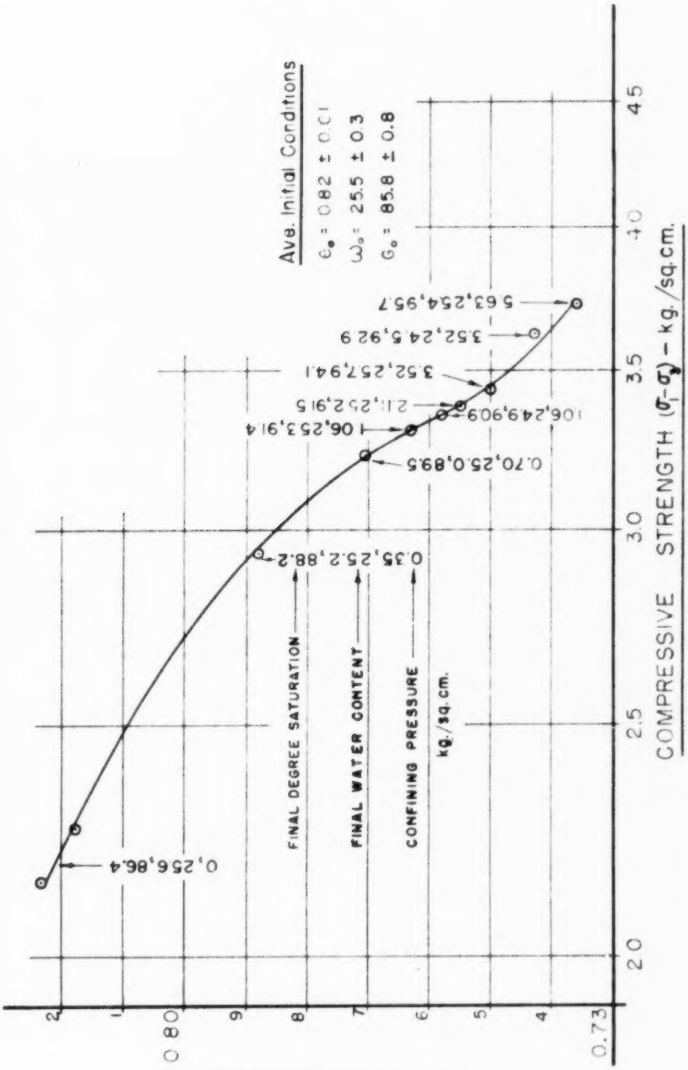
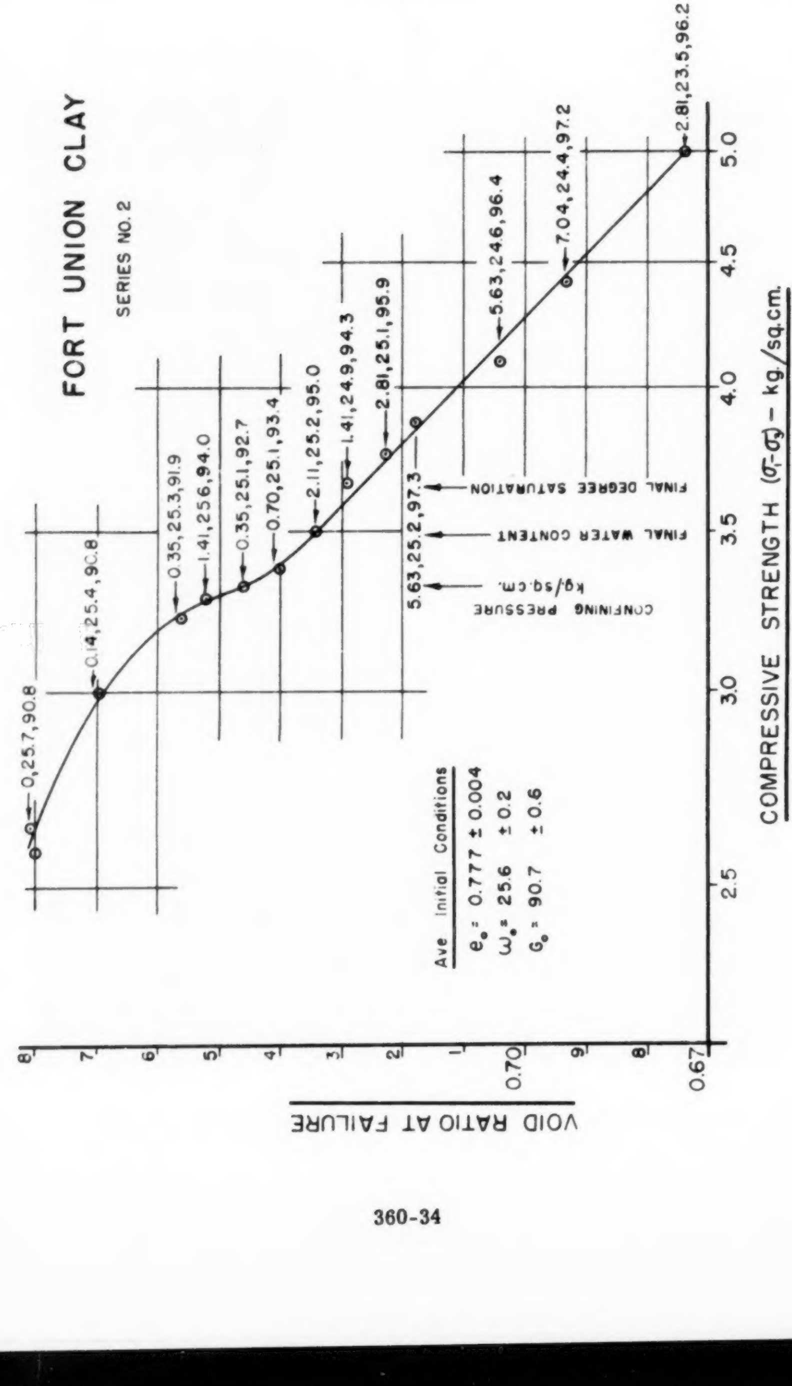

FORT UNION CLAY
SERIES NO. 1
Ave. Initial Conditions
e₀ = 0.82 ± 0.01
ω₀ = 25.5 ± 0.3
G₀ = 85.8 ± 0.8
FINAL DEGREE SATURATION
FINAL WATER CONTENT
CONFINING PRESSURE
kg./sq.cm.
0,25.6,86.4
0.35,25.2,88.2
0.70,25.0,89.5
1.06,25.3,91.4
1.06,24.9,90.9
2.11,25.2,91.5
3.52,25.7,94.1
3.52,24.5,92.9
5.63,25.4,95.7
VOID RATIO AT FAILURE
COMPRESSIVE STRENGTH (σ1-σ3) - kg./sq.cm.

Figure 13



FORT UNION CLAY
SERIES NO. 2
0,25.7,90.8
0.14,25.4,90.8
0.35,25.3,91.9
1.41,25.6,94.0
0.35,25.1,92.7
0.70,25.1,93.4
2.11,25.2,95.0
1.41,24.9,94.3
2.81,25.1,95.9
5.63,25.2,97.3
5.63,24.6,96.4
7.04,24.4,97.2
2.81,23.5,96.2
CONFINING PRESSURE kg/sq.cm.
FINAL WATER CONTENT
FINAL DEGREE SATURATION
Ave Initial Conditions
$e_o$ = 0.777 ± 0.004
$\omega_o$ = 25.6 ± 0.2
$G_o$ = 90.7 ± 0.6
VOID RATIO AT FAILURE
0.70
0.67
COMPRESSIVE STRENGTH $(\sigma_1-\sigma_3)$ – kg./sq.cm.
2.5
3.0
3.5
4.0
4.5
5.0

Figure 14

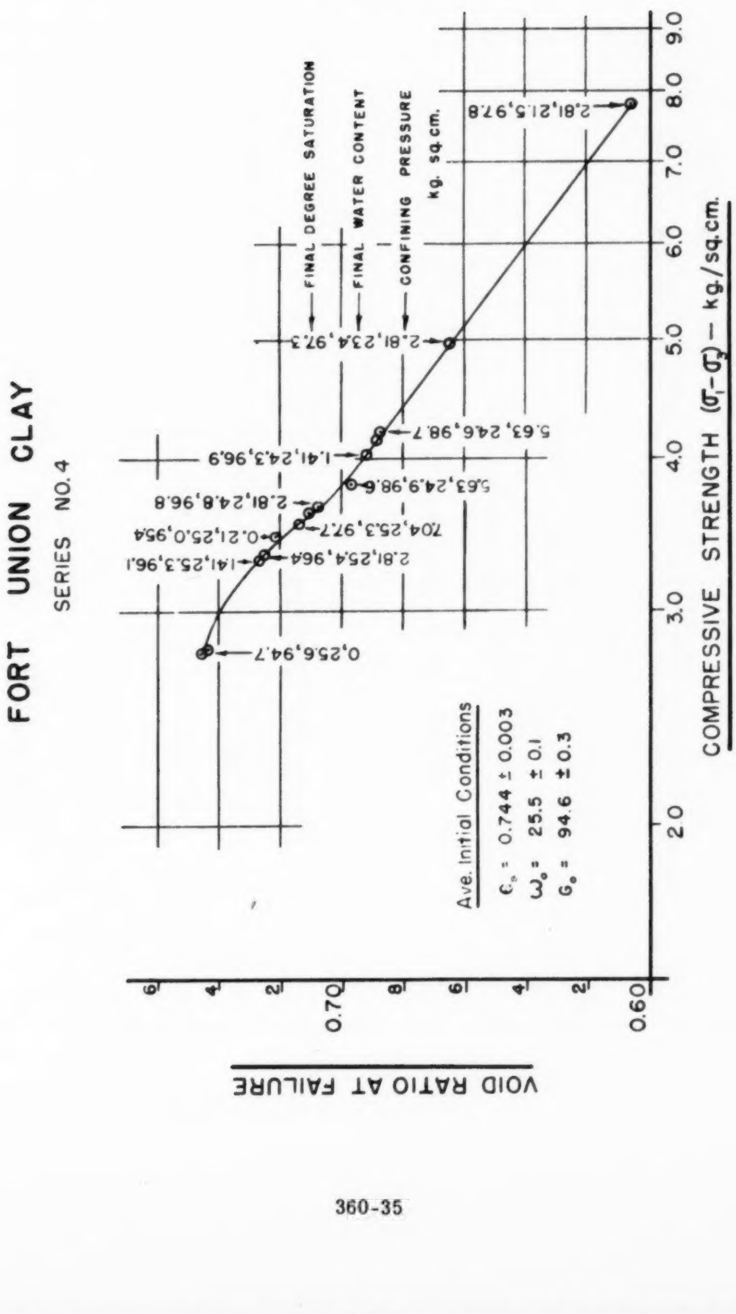
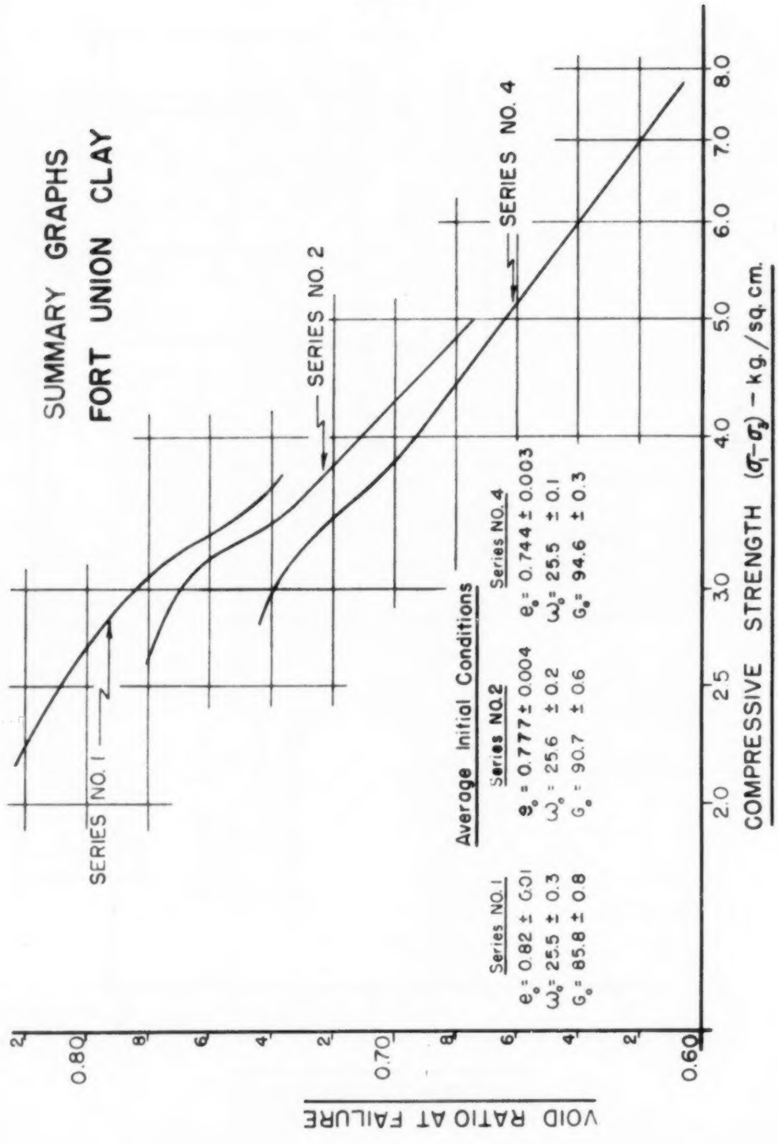

FORT UNION CLAY
SERIES NO.4
FINAL DEGREE SATURATION
FINAL WATER CONTENT
CONFINING PRESSURE
kg. sq.cm.
2.81,21.5,97.8
2.81,23.4,97.3
5.63,24.6,98.7
1.41,24.3,96.9
5.63,24.9,98.6
2.81,24.8,96.8
7.04,25.3,97.7
0.21,25.0,95.4
2.81,25.4,96.4
1.41,25.3,96.1
0,25.6,94.7
Ave. Initial Conditions
$e_o$ = 0.744 ± 0.003
$\omega_o$ = 25.5 ± 0.1
$G_o$ = 94.6 ± 0.3
VOID RATIO AT FAILURE
0.60
0.70
COMPRESSIVE STRENGTH $(\sigma_1 - \sigma_3)$ – kg./sq.cm.
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0

Figure 15



SUMMARY GRAPHS
FORT UNION CLAY
SERIES NO. 1
SERIES NO. 2
SERIES NO. 4
Average Initial Conditions
Series NO. 1
$e_o$ = 0.82 ± 0.01
$\omega_o$ = 25.5 ± 0.3
$G_o$ = 85.8 ± 0.8
Series NO. 2
$e_o$ = 0.777 ± 0.004
$\omega_o$ = 25.6 ± 0.2
$G_o$ = 90.7 ± 0.6
Series NO. 4
$e_o$ = 0.744 ± 0.003
$\omega_o$ = 25.5 ± 0.1
$G_o$ = 94.6 ± 0.3
VOID RATIO AT FAILURE
0.60
0.70
0.80
COMPRESSIVE STRENGTH $(\sigma_1-\sigma_3)$ – kg./sq.cm.
2.0
2.5
3.0
4.0
5.0
6.0
7.0
8.0

Figure 16

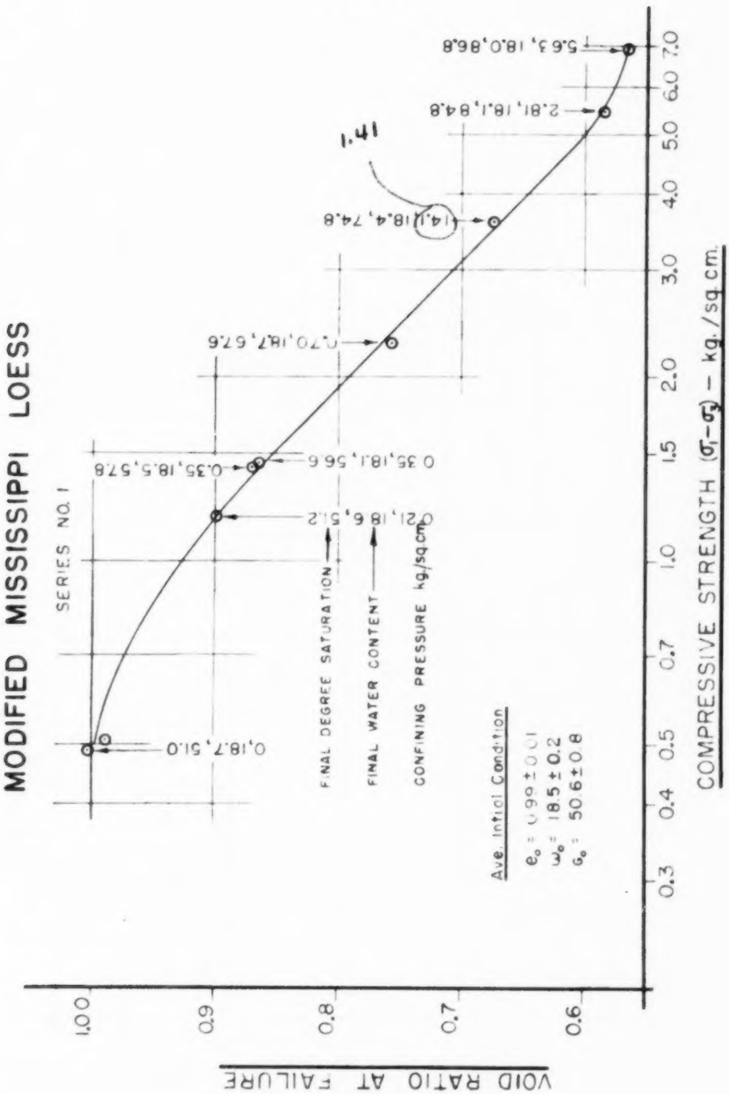

MODIFIED MISSISSIPPI LOESS
SERIES NO. 1
FINAL DEGREE SATURATION
FINAL WATER CONTENT
CONFINING PRESSURE kg./sq.cm.
Ave. Initial Condition
VOID RATIO AT FAILURE
COMPRESSIVE STRENGTH $(\sigma_1-\sigma_3)$ – kg./sq.cm.
1.41

**Figure 17**

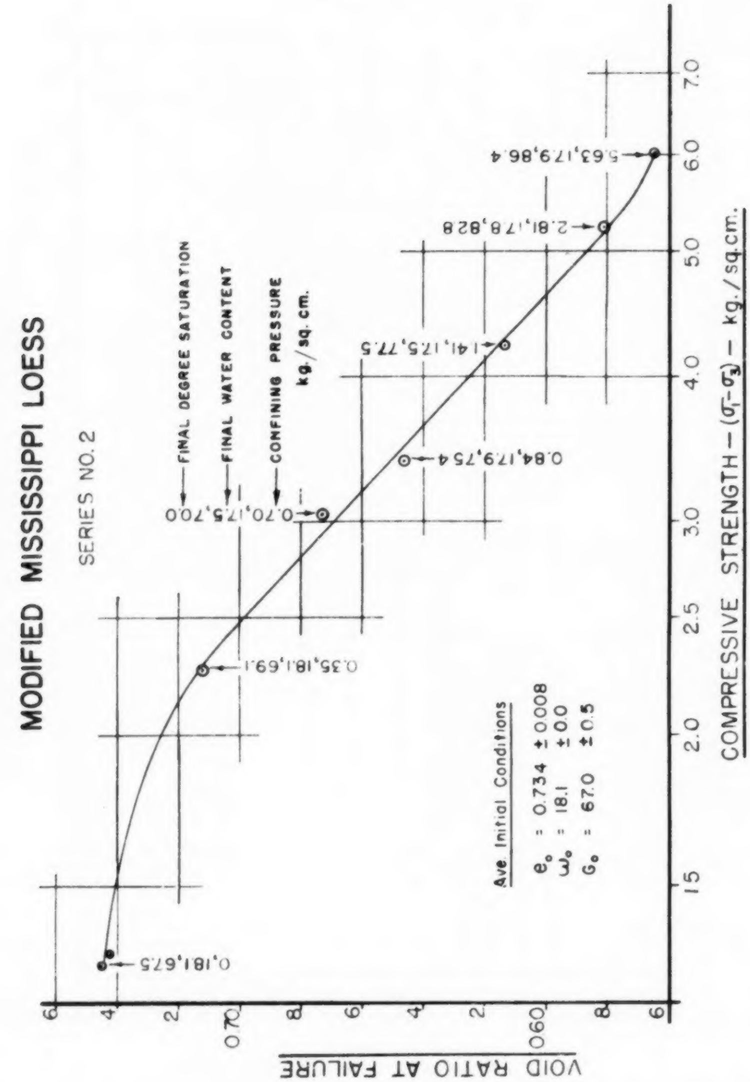
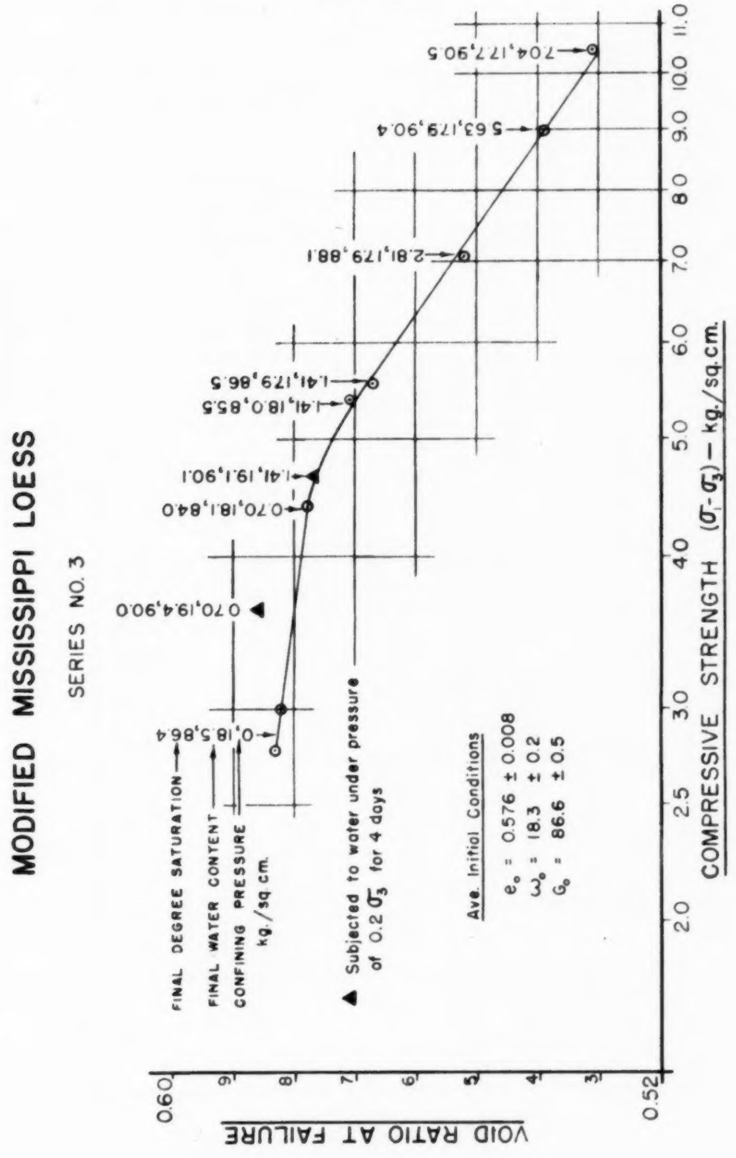

MODIFIED MISSISSIPPI LOESS
SERIES NO. 2
FINAL DEGREE SATURATION
FINAL WATER CONTENT
CONFINING PRESSURE kg./sq. cm.
Ave. Initial Conditions
$e_o$ = 0.734 ± 0.008
$\omega_o$ = 18.1 ± 0.0
$G_o$ = 67.0 ± 0.5
COMPRESSIVE STRENGTH — $(\sigma_1 - \sigma_3)$ — kg./sq.cm.
VOID RATIO AT FAILURE

Figure 18



MODIFIED MISSISSIPPI LOESS
SERIES NO. 3
FINAL DEGREE SATURATION
FINAL WATER CONTENT
CONFINING PRESSURE kg./sq.cm.
Subjected to water under pressure of 0.2 $\sigma_3$ for 4 days
Ave. Initial Conditions
$e_o$ = 0.576 ± 0.008
$\omega_o$ = 18.3 ± 0.2
$G_o$ = 86.6 ± 0.5
0,18.5,86.4
0.70,19.4,90.0
0.70,18.1,84.0
1.41,19.1,90.1
1.41,18.0,85.5
1.41,17.9,86.5
2.81,17.9,88.1
5.63,17.9,90.4
7.04,17.7,90.5
VOID RATIO AT FAILURE
0.60
9
8
7
6
5
4
3
0.52
COMPRESSIVE STRENGTH $(\sigma_1-\sigma_3)$ — kg./sq.cm.
2.0
2.5
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0

Figure 19

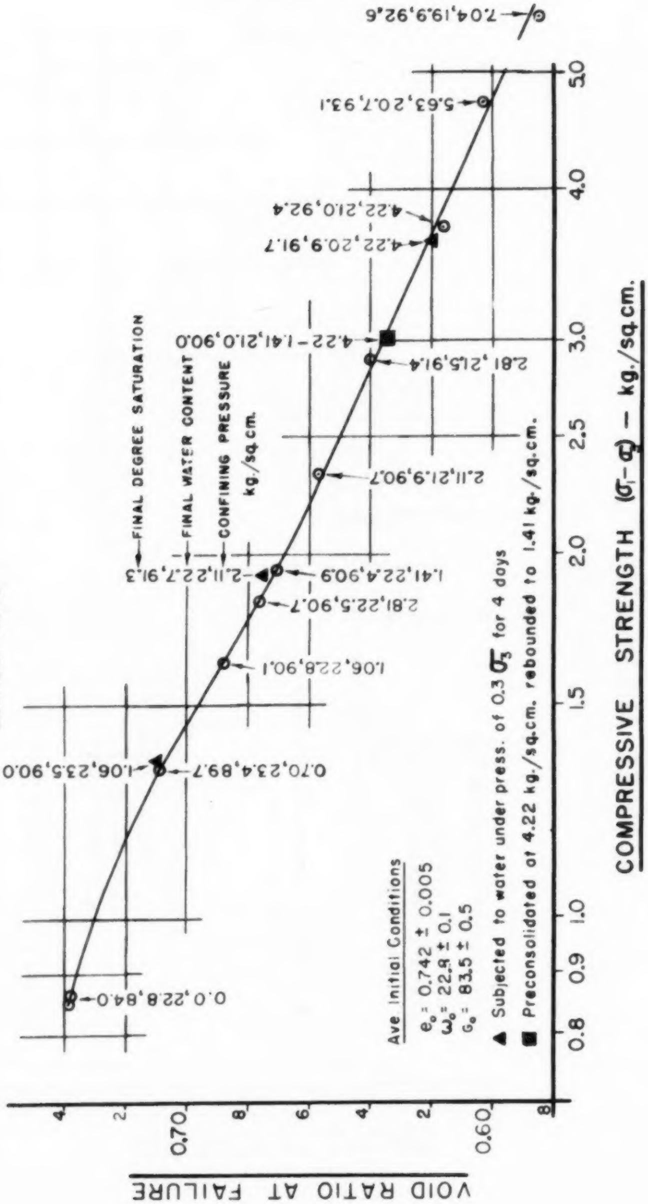

MODIFIED MISSISSIPPI LOESS
SERIES NO. 5
FINAL DEGREE SATURATION
FINAL WATER CONTENT
CONFINING PRESSURE
kg./sq.cm.
Ave Initial Conditions
$e_o$ = 0.742 ± 0.005
$\omega_o$ = 22.8 ± 0.1
$G_o$ = 83.5 ± 0.5
▲ Subjected to water under press. of 0.3 $\sigma_3$ for 4 days
■ Preconsolidated at 4.22 kg./sq.cm. rebounded to 1.41 kg./sq.cm.
0.0, 22.8, 84.0
0.70, 23.4, 89.7
1.06, 23.5, 90.0
1.06, 22.8, 90.1
2.81, 22.5, 90.7
1.41, 22.4, 90.9
2.11, 22.7, 91.3
2.11, 21.9, 90.7
2.81, 21.5, 91.4
4.22 - 1.41, 21.0, 90.0
4.22, 20.9, 91.7
4.22, 21.0, 92.4
5.63, 20.7, 93.1
7.04, 19.9, 92.6
VOID RATIO AT FAILURE
COMPRESSIVE STRENGTH $(\sigma_1 - \sigma_3)$ – kg./sq.cm.
0.8
0.9
1.0
1.5
2.0
2.5
3.0
4.0
5.0
0.60
0.70

Figure 20

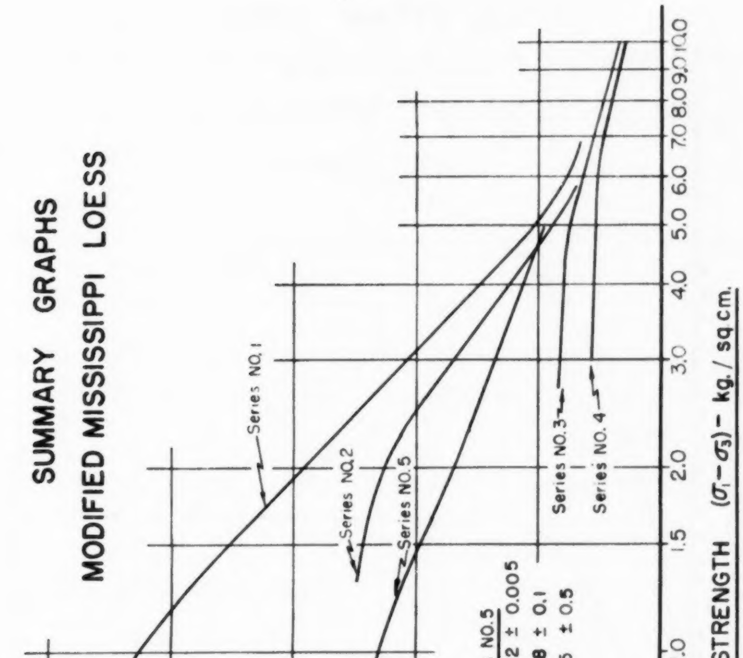

SUMMARY GRAPHS
MODIFIED MISSISSIPPI LOESS
Series NO. 1
$e_o$ = 0.99 ± 0.01
$\omega_o$ = 18.5 ± 0.2
$G_o$ = 50.6 ± 0.8
Series NO. 2
$e_o$ = 0.734 ± 0.008
$\omega_o$ = 18.1 ± 0.0
$G_o$ = 67.0 ± 0.5
Series NO. 3
$e_o$ = 0.576 ± 0.008
$\omega_o$ = 18.3 ± 0.2
$G_o$ = 86.6 ± 0.5
Series NO. 4
$e_o$ = 0.545 ± 0.007
$\omega_o$ = 18.0 ± 0.1
$G_o$ = 90.0 ± 0.6
Series NO. 5
$e_o$ = 0.742 ± 0.005
$\omega_o$ = 22.8 ± 0.1
$G_o$ = 83.5 ± 0.5
Series NO. 1
Series NO. 2
Series NO. 5
Series NO. 3
Series NO. 4
VOID RATIO AT FAILURE
1.00
.90
.80
.70
.60
.50
0.3
0.4
0.5
0.7
1.0
1.5
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
COMPRESSIVE STRENGTH $(\sigma_1 - \sigma_3)$ - kg./sq cm.

Figure 21

# STRESS-STRAIN CURVES

FORT UNION CLAY

SERIES NO. 4

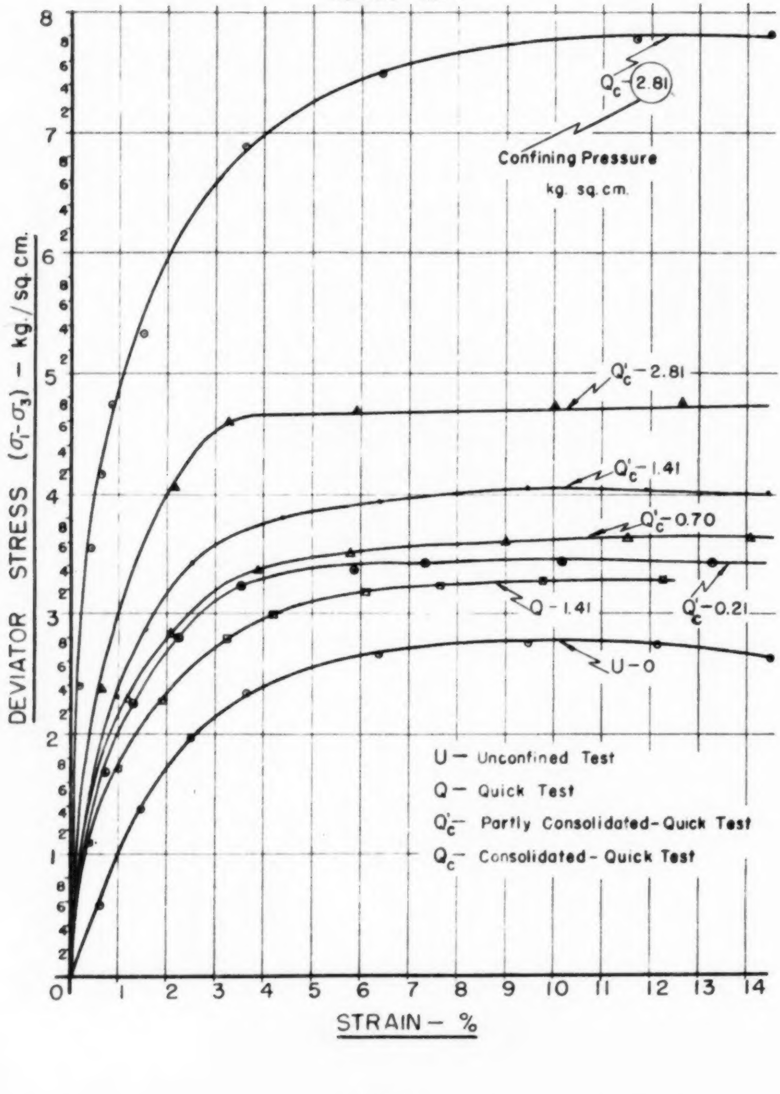
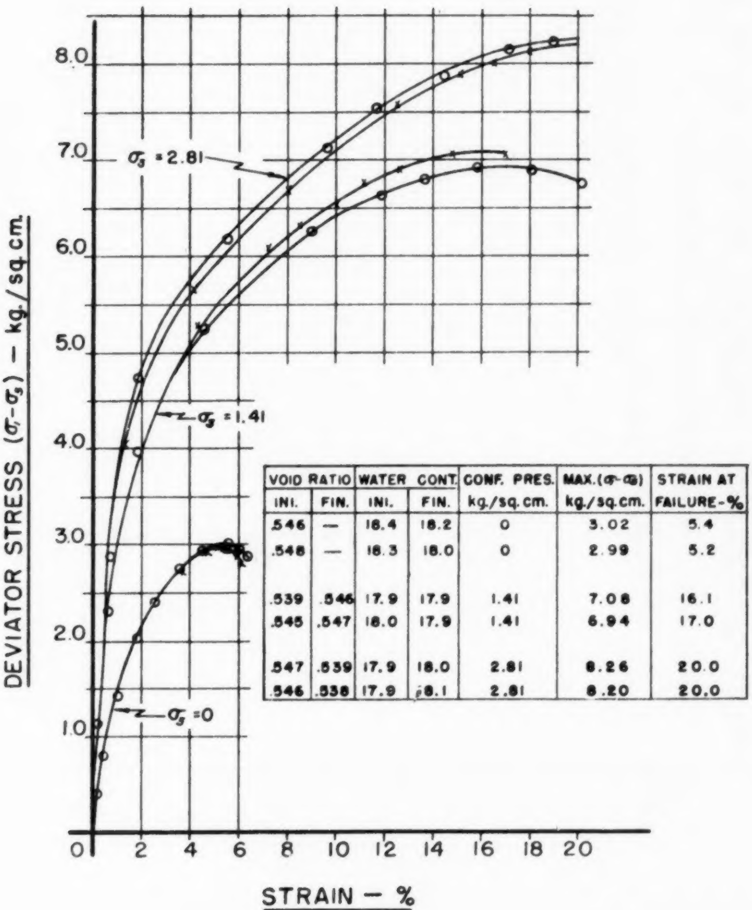


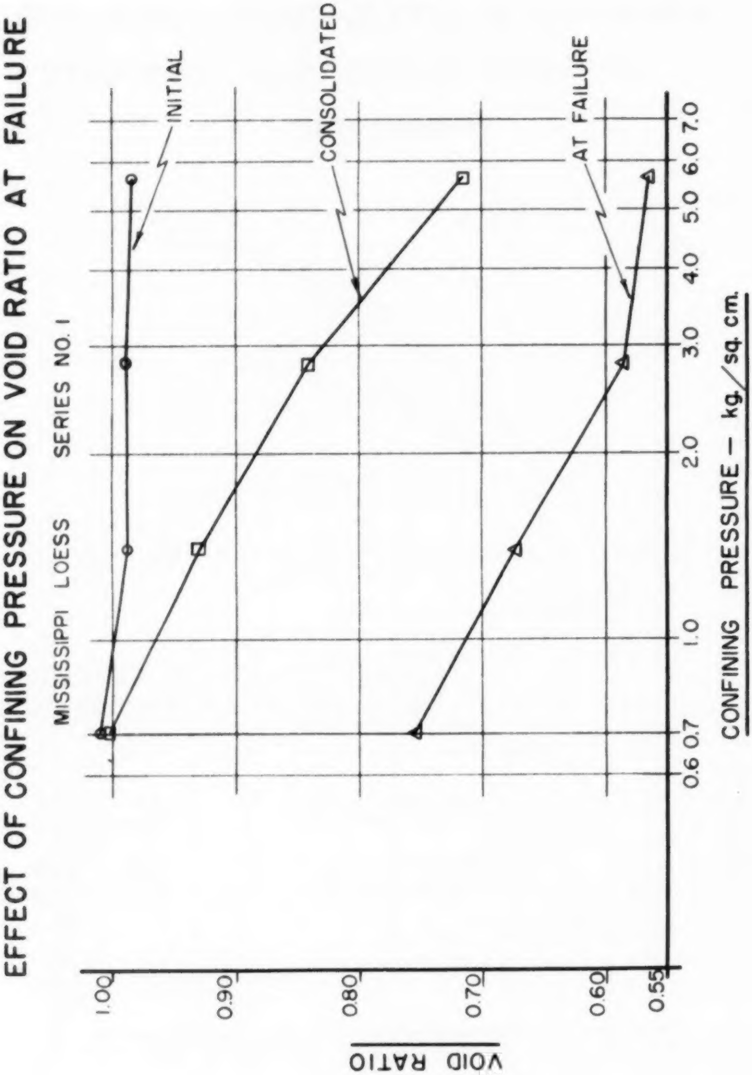
Figure 22

# COMPARISON OF STRESS-STRAIN CURVES FOR SPECIMENS TESTED UNDER DUPLICATE CONDITIONS



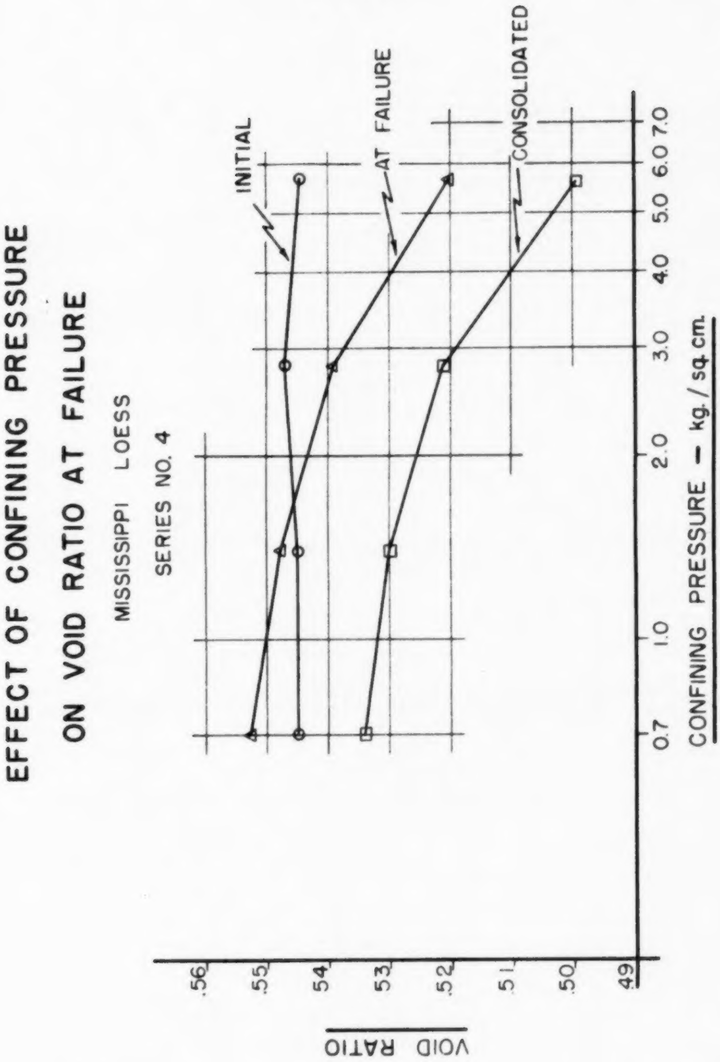

| VOID RATIO | | WATER CONT. | | CONF. PRES. | MAX. $(\sigma_1-\sigma_3)$ | STRAIN AT |
|---|---|---|---|---|---|---|
| INI. | FIN. | INI. | FIN. | kg./sq. cm. | kg./sq. cm. | FAILURE-% |
| .546 | — | 18.4 | 18.2 | 0 | 3.02 | 5.4 |
| .548 | — | 18.3 | 18.0 | 0 | 2.99 | 5.2 |
| .539 | .546 | 17.9 | 17.9 | 1.41 | 7.08 | 16.1 |
| .545 | .547 | 18.0 | 17.9 | 1.41 | 6.94 | 17.0 |
| .547 | .539 | 17.9 | 18.0 | 2.81 | 8.26 | 20.0 |
| .546 | .538 | 17.9 | 18.1 | 2.81 | 8.20 | 20.0 |

Figure 23



EFFECT OF CONFINING PRESSURE ON VOID RATIO AT FAILURE
MISSISSIPPI LOESS SERIES NO. 1
INITIAL
CONSOLIDATED
AT FAILURE
VOID RATIO
1.00
0.90
0.80
0.70
0.60
0.55
0.6 0.7
1.0
2.0
3.0
4.0
5.0
6.0 7.0
CONFINING PRESSURE — kg/sq. cm.

Figure 24



EFFECT OF CONFINING PRESSURE
ON VOID RATIO AT FAILURE
MISSISSIPPI LOESS
SERIES NO. 4
INITIAL
AT FAILURE
CONSOLIDATED
.56
.55
.54
.53
.52
.51
.50
.49
VOID RATIO
0.7
1.0
2.0
3.0
4.0
5.0
6.0
7.0
CONFINING PRESSURE — kg. / sq. cm.

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